Northeastern University
Library

NATIONAL ASSOCIATION

OF

CEMENT USERS

PROCEEDINGS

OF THE

NINTH ANNUAL CONVENTION

Held at Pittsburgh, Pa.

December 10, 11, 12, 13, 14, 1912

VOLUME IX

EDITED BY RICHARD L. HUMPHREY

PUBLISHED BY THE INSTITUTE

1917

Sci 1520.485
Stack
HARVARD COLLEGE
DEC 1929
LIBRARY
Gift of the
Engineering School.

COPYRIGHT 1917 BY AMERICAN CONCRETE INSTITUTE

GORDON McKAY LIBRARY
JUN 15 1970
HARVARD UNIVERSITY

DISCARDED BY
GORDON McKAY LIBRARY

The Institute is not responsible, as a body, for the statements and opinions advanced in its publications.

CONTENTS.

PAGE

Officers 5
Sectional Committees 6
Past Officers 8
Charter 9
By-Laws 10
Summary of the Proceedings of the Ninth Annual Convention 15
Annual Address by the President, The Use of Concrete in Mines — Richard L. Humphrey 27
Effects of Electric Currents on Concrete—E. B. Rosa, Burton McCollum, O. S. Peters 45
Discussion 107
Report of the Committee on Reinforced Concrete and Building Laws —Alfred E. Lindau, Chairman 112
Principles of Design and Results of Tests on Girderless Floor Construction of Reinforced Concrete—T. L. Condron 116
Measurement of Actual Stresses in a Cantilever Flat Slab Reinforced Concrete Floor having Rectangular Panels—Arthur R. Lord 127
Discussion 151
Design of Rigid Frames in Steel and Reinforced Concrete—Sanford E. Thompson and Edward Smulski 156
Tests to Determine Lateral Distribution of Stresses in Wide Reinforced Concrete Beams—W. A. Slater 182
Discussion 215
The Present Status of Unit Methods of Reinforced Concrete Construction—John E. Conzelman 218
Report of the Committee on Concrete Surfaces, presenting Standard Specifications for Portland Cement Stucco—L. C. Wason, Chairman 233
Discussion 242
Tests of Waterproofing for Concrete—Cloyd M. Chapman 250
The Use of Cement on National Irrigation Works—Arthur P. Davis 258
The Use of Concrete by the Bureau of Yards and Docks, U. S. Navy —H. R. Stanford 266
Concrete in Railroad Work—M. A. Long 277
Application of Concrete in the Abolishment of Grade Crossings in the City of Philadelphia—James W. Phillips 285
Some Recent Applications of Concrete in Railroad Work—Frederick Auryansen 293
Recent Applications of Concrete and Reinforced Concrete in the City of Pittsburgh—N. S. Sprague 307
Discussion 324

PAGE

The Development of Concrete Grain Elevator Construction—R. P. Durham 326
Concrete Elevators—Barney I. Weller 338
Report of the Committee on Measuring Concrete, presenting Standard Methods for the Measurement of Concrete Work—Robert A. Cummings, Chairman 348
Discussion 355
Report of the Committee on Specifications and Methods of Tests for Concrete Materials—Sanford E. Thompson, Chairman 356
Discussion 366
The Constitution of Portland Cement. Some Results Obtained at the Experimental Cement Plant of the Bureau of Standards—P. H. Bates 368
Discussion 378
The Progress of the Work of the Bureau of Standards in Cement and Concrete—Rudolph J. Wig 380
Sand and Gravel Washing Plants—Raymond W. Dull 393
Topical Discussion on Concrete Floors, Sidewalks, Roads, Highway Bridges and Culverts 404
Report of the Committee on Reinforced Concrete Highway Bridges and Culverts—Willis Whited, Chairman 412
Discussion 417
Report of the Committee on Roadways, Sidewalks and Floors—C. W. Boynton, Chairman 427
Standard Specifications for Plain Concrete Floors 431
Standard Specifications for Reinforced Concrete Floors 437
Standard Specifications for Concrete Curb and Concrete Curb and Gutter 442
Standard Specifications for Portland Cement Sidewalks 449
Standard Specifications for Concrete Roads and Street Pavements 455
The Concrete Roads of Wayne County, Michigan—Edward N. Hines 462
Discussion 470
Vibrolithic Concrete Pavements—R. C. Stubbs 476
The Manufacture of Concrete Products in Germany—Julius Carstanjen, Jr. 480
Discussion 488
The Coloring and Texture of Concrete Products—Adolph Schilling 491
Reinforced Concrete Silos and Small Grain Bins—E. S. Fowler 498
Annual Report of the Executive Board 511
Minutes of Meetings of the Executive Board 518
Register of Members in Attendance—Ninth Convention 523
Subject Index 525
Author Index 533
List of Publications 537

PLATES.

I. Fig. 15.—Reinforced Concrete Bridge, Trestle and Coal Tipple, at Floreffe, Belgium ..... (opposite) 44

OFFICERS

OF

THE NATIONAL ASSOCIATION OF CEMENT USERS

PRESIDENT.

RICHARD L. HUMPHREY.

FIRST VICE-PRESIDENT.

EDWARD D. BOYER.

SECOND VICE-PRESIDENT.

ARTHUR N. TALBOT.

THIRD VICE-PRESIDENT.

EDWARD S. LARNED.

FOURTH VICE-PRESIDENT.

IRA H. WOOLSON.

SECRETARY.

EDWARD E. KRAUSS.

TREASURER.

HENRY C. TURNER.

SECTIONAL VICE-PRESIDENTS.

(See page 6.)

SECTIONAL COMMITTEES.

(The Chairmen are Vice-Presidents of the Association.)

AGGREGATES.

WILLIAM M. KINNEY, *Chairman.*

F. W. KELLEY,
H. L. SHERMAN,
R. J. WIG,
M. O. WITHEY.

BUILDING BLOCKS AND CEMENT PRODUCTS.

P. S. HUDSON,* *Chairman.*

C. K. ARP,
P. H. ATWOOD,
ROBERT F. HAVLIK,
CHARLES D. WATSON.

EDUCATION.

LOGAN WALLER PAGE, *Chairman.*
PERCY H. WILSON, *Secretary.*

A. C. TRUE,
WILLIAM G. HARTRANFT,
W. A. HOLMAN,
MORRIS METCALF,
A. MOYER,
J. P. H. PERRY.

EXHIBITION.

H. S. DOYLE, *Chairman.*

B. F. AFFLECK,
W. H. BURKE,
W. J. ROSEBERRY, JR.,
P. AUSTIN TOMES.

FIREPROOFING.

RUDOLPH P. MILLER, *Chairman.*

EDWIN CLARK,
W. C. ROBINSON,
JOHN STEPHEN SEWELL,
IRA H. WOOLSON.

INSURANCE.

WILLIAM H. HAM, *Chairman.*

A. L. JOHNSON,
W. H. MERRILL,
F. W. MOSES,
J. P. H. PERRY.

* Died November 26, 1912.

Sectional Committees.

MEASURING CONCRETE.

Robert A. Cummings, *Chairman.*

L. H. ALLEN,
ROBERT ANDERSON,
CHARLES DERLETH, JR.,
THOMAS M. VINTON.

NOMENCLATURE.

Peter Gillespie, *Chairman.*

E. P. GOODRICH,
E. J. MEHREN,
F. E. TURNEAURE,
FRANK C. WIGHT.

REINFORCED CONCRETE AND BUILDING LAWS.

Alfred E. Lindau, *Chairman.*

W. P. ANDERSON,
E. J. MOORE,
ARTHUR N. TALBOT,
SANFORD E. THOMPSON.

ROADWAYS, SIDEWALKS AND FLOORS.

C. W. Boynton, *Chairman.*

A. C. BIRNIE,
J. B. LANDFIELD,
C. R. MILLER,
A. E. SNODGRASS.

SPECIFICATIONS AND METHODS OF TESTS FOR CONCRETE MATERIALS.

Sanford E. Thompson, *Chairman.*

CLOYD M. CHAPMAN,
WILLIAM B. FULLER,
RUSSELL S. GREENMAN,
ARTHUR N. TALBOT.

TREATMENT OF CONCRETE SURFACES.

Leonard C. Wason, *Chairman.*

CLOYD M. CHAPMAN,
ALFRED HOPKINS,
EMILE G. PERROT,
H. H. QUIMBY.

PAST OFFICERS.

President.	1905	John P. Given. (Presiding Officer First Convention.)
	1905–	Richard L. Humphrey.
First Vice-President.	1905	A. L. Goetzmann.
	1906–9	Merrill Watson.
	1909–12	Edward D. Boyer.
Second Vice-President.	1905–6	John H. Fellows.
	1907–10	M. S. Daniels.
	1911–12	Arthur N. Talbot.
Third Vice-President.	1905	H. C. Quinn.
	1906–7	O. U. Miracle.
	1908	S. B. Newberry.
	1909–12	E. S. Larned.
Fourth Vice-Presidents.	1905–7	A. Monsted.
	1908–9	George C. Walters.
	1909–10	F. A. Norris.
	1911–12	Ira H. Woolson.
Treasurer.	1905	A. S. J. Gammon.
	1906–12	H. C. Turner.
Secretary.	1905–6	Charles C. Brown.
	1907	W. W. Curtis.
	1908–9	George C. Wright.
	1910	Edward E. Krauss (Acting)
	1911–	Edward E. Krauss.

CHARTER

OF

THE NATIONAL ASSOCIATION OF CEMENT USERS.

KNOW ALL MEN BY THESE PRESENTS, That we, the undersigned, all of whom are citizens of the United States, and a majority of whom are residents of the District of Columbia, have associated ourselves together for the purpose hereinafter set forth and desiring that we may be incorporated as an Association under sub-chapter three (3) of the Incorporation Laws of the District of Columbia, as provided in the Code of Law of the District of Columbia, enacted by Congress and approved by the President of the United States, do hereby certify:

1. Name. The name of the proposed corporation is "The National Association of Cement Users."

2. Term of Existence. The existence of the said corporation shall be perpetual.

3. Objects. The particular business and objects of the said corporation shall be to disseminate information and experience upon and to promote the best methods to be employed in the various uses of cement by means of convention, the reading and discussion of papers upon materials of a cement nature and their uses, by social and friendly intercourse at such conventions, the exhibition and study of materials, machinery and methods and to circulate among its members by means of publications the information thus obtained.

4. Incorporators. The number of its managers for the first year shall be fifteen.

In Witness Whereof, we have hereunto set our hands and seals this fourteenth day of December, A. D. 1906.

RICHARD L. HUMPHREY, (SEAL)
JOHN STEPHEN SEWELL, (SEAL)
S. S. VOORHEES. (SEAL)

OFFICE OF RECORDER OF DEEDS,
DISTRICT OF COLUMBIA.

This is to certify that the foregoing is a true and verified copy of a Certificate of Incorporation, and of the whole of such Certificate as received for record in this office at 9.49 A. M., the 19th day of December, A. D. 1906.

In testimony whereof I have hereunto set my hand and affixed the seal of this office, this 20th day of December, A. D. 1906.

(Signed) R. W. DUTTON,
Deputy Recorder of Deeds,
District of Columbia.

(SEAL)

BY-LAWS.

Article I.

MEMBERS.

Section 1. Any person engaged in the construction or maintenance of work in which cement is used, or qualified by business relations or practical experience to co-operate in the purposes of this Association, or engaged in the manufacture or sale of machinery or supplies for cement users, or a man who has attained eminence in the field of engineering, architecture or applied science, is eligible for membership.

Sec. 2. A firm or company shall be treated as a single member.

Sec. 3. Any member contributing annually twenty or more dollars in addition to the regular dues shall be designated and listed as a Contributing Member.

Sec. 4. Application for membership shall be made to the Secretary on a form prescribed by the Board of Direction. The Secretary shall submit monthly or oftener if necessary to each member of the Board of Direction for letter ballot a list of all applicants for membership on hand at that time with a statement of the qualifications, and a two-thirds majority of the members of the Board shall be necessary to an election.

Applicants for membership shall be qualified upon notification of election by the Secretary by the payment of the annual dues, and unless these dues are paid within 60 days thereafter the election shall become void. An extract of the By-Laws relating to dues shall accompany the notice of election.

Sec. 5. Resignations from membership must be presented in writing to the Secretary on or before the close of the fiscal year and shall be acceptable provided the dues are paid for that year.

Article II.

OFFICERS.

Section 1. The officers shall be the President, two Vice-Presidents, six Directors (one from each geographical district), the Secretary and the Treasurer, who, with the five latest living Past-Presidents, who continue to be members, shall constitute the Board of Direction.

Sec. 2. The Board of Direction shall, from time to time, divide the territory occupied by the membership into six geographical districts, to be designated by numbers.

Sec. 3. The terms of office of the President, Secretary and Treasurer shall be one year; of the Vice-Presidents and the Directors, two years. Provided, however, that at the first election after the adoption of this By-Law, a President, one Vice-President, three Directors and a Treasurer shall be elected to serve for one year only, and one Vice-President and three Directors for two years; provided, also, that after the first election a President, one Vice-President, three Directors and a Treasurer shall be elected annually.

The term of each officer shall begin at the close of the Annual Convention at which such officer is elected, and shall continue for the period above named or until a successor is duly elected.

A vacancy in the office of President shall be filled by the senior Vice-President. A vacancy in the office of Vice-President shall be filled by the senior Director.

Seniority between persons holding similar offices shall be determined by priority of election to the office, and when these dates are the same, by priority of admission to membership; and when the latter dates are identical, the selection shall be made by lot. In case of the disability or neglect in the performance of his duty, of any officer of this Association, the Board of Direction shall have power to declare the office vacent. Vacancies in any office for the unexpired term shall be filled by the Board of Direction, except as provided above.

Sec. 4. The Board of Direction shall appoint the Secretary; it shall create such special committees as may be deemed desirable for the purpose of preparing recommended practice and standards concerning the proper use of cement for consideration by the

Association, and shall appoint a chairman for each committee. Four or more additional members on each special committee shall be appointed by the President, in consultation with the Chairman.

SEC. 5. It shall be the duty of the Board of Direction to audit the accounts of the Secretary and the Treasurer before each annual convention.

SEC. 6. The Board of Direction shall appoint a Committee on Nomination of Officers and a Committee on Resolutions, to be announced by the President at the first regular session of the annual convention.

SEC. 7. There shall be an Executive Committee of the Board of Direction, consisting of the President, the Secretary, the Treasurer and two of its members, appointed by the Board of Direction.

SEC. 8. The Executive Committee shall manage the affairs of the Association during the interim between the meetings of the Board of Direction.

SEC. 9. The President shall have general supervision of the affairs of the Association. He shall preside at the Annual Convention, at the meetings of the Board of Direction and the Executive Committee, and shall be ex-officio member of all committees.

The Vice-Presidents in order of seniority shall discharge the duties of the President in his absence.

SEC. 10. The Secretary shall perform such duties and furnish such bond as may be determined by the Board of Direction.

SEC. 11. The Treasurer shall be the custodian of the funds of the Association, shall disburse the same in the manner prescribed and shall furnish bond in such sum as the Board of Direction may determine.

SEC. 12. The Secretary shall receive such salary as may be fixed by the Board of Direction.

ARTICLE III.

MEETINGS.

SECTION 1. The Association shall meet annually. The time and place shall be fixed by the Board of Direction and notice of this action shall be mailed to all members at least thirty days previous to the date of the Convention.

Sec. 2. The Board of Direction shall meet immediately after the Convention at which it was elected, effect organization and transact such business as may be necessary.

Sec. 3. The Board of Direction shall meet at least twice each year. The time and place to be fixed by the Executive Committee.

Sec. 4. A majority of the members shall constitute a quorum for meetings of the Board of Direction and of the Executive Committee.

Article IV.

DUES.

Section 1. The fiscal year shall commence on the first of July and all dues shall be payable in advance.

Sec. 2. The annual dues of each member shall be five dollars ($5.00).

Sec. 3. Any person elected after six months of any fiscal year shall have expired, need pay only one-half of the amount of dues for that fiscal year; but he shall not be entitled to a copy of the Proceedings of that year.

Sec. 4. A member whose dues remain unpaid for a period of three months shall forfeit the privilege of membership and shall be officially notified to this effect by the Secretary, and if these dues are not paid within thirty days thereafter his name shall be stricken from the list of members. Members may be reinstated upon the payment of all indebtedness against them upon the books of the Association.

Article V.

RECOMMENDED PRACTICE AND SPECIFICATIONS.

Section 1. Proposed Recommended Practice and Specifications to be submitted to the Association must be mailed to the members at least thirty days prior to the Annual Convention, and as there amended and approved, passed to letter ballot, which shall be canvassed within sixty days thereafter; such Recommended Practice and Specifications shall be considered adopted unless at least ten per cent of the total membership shall vote in the negative.

Article VI.

AMENDMENTS.

Section 1. Amendments to these By-Laws, signed by at least fifteen members, must be presented in writing to the Board of Direction ninety days before the Annual Convention and shall be printed in the notice of the Annual Convention. These amendments may be discussed and amended at the Annual Convention and passed to letter ballot by a two-thirds vote of those present. Two-thirds of the votes cast by letter ballot shall be necessary for their adoption.

SUMMARY OF THE PROCEEDINGS OF THE NINTH ANNUAL CONVENTION.

FIRST SESSION, TUESDAY, DECEMBER 10, 1912, 8 P. M.

The Convention was called to order by the President, Richard L. Humphrey.

The President introduced Charles J. O'Brien, City Solicitor, who delivered an address of welcome to the City of Pittsburgh on behalf of the Mayor, the Honorable William A. Magee, in part as follows:

I am here this evening to represent His Honor the Mayor, to present his apologies for his inability to attend, and to extend to your Association the greetings of the City.

We are glad to welcome so important a body as this to Pittsburgh; we trust you will find it a place of interest and entertainment.

This great industrial center has commenced a series of public improvements of great magnitude; the largest ever undertaken by it; and in its work it is being greatly aided by its various civic bodies. We trust that you will find time to go about Pittsburgh and see what it is; especially compare it as it is now with what it was a few years ago and be convinced of the fact that it is improving in all directions. Not only has it extended as an industrial and commercial center, but its municipal improvements have kept it abreast with the great cities of this country; its people are progressive in the truest and best sense of the word—I think you will find evidence of this on all sides.

Now, gentlemen, in conclusion let me say that you are indeed welcome within our gates. We trust you will enjoy yourselves, visit our art galleries and other places of public interest, and make yourselves truly at home. We believe that when you have spent a few days in our midst you will be entirely satisfied with the selection you have made of a place of meeting for your Association, and we trust also that your deliberations will be entirely harmonious and successful.

The President introduced Julius Carstanjen, Jr., of Duisburg, Germany, who presented greetings on behalf of the Deutscher Beton-Verein as follows:

It is a great pleasure indeed to have the honor, as the son of the senior member of the Executive Board of the German Concrete Association, of being

directed by the President, Mr. Alfred Huser, to present to you most hearty greetings from the German Concrete Association and best wishes for a most successful convention.

An address of welcome on behalf of the Architects was made by Edward Stotz, President of the Pennsylvania State Association, American Institute of Architects, who spoke as follows:

It is indeed an honor and a very pleasant duty to welcome you to the State of Pennsylvania and the City of Pittsburgh on behalf of the architects, and I regret very much that many of the members of our profession who would otherwise have been here, are unavoidably absent on account of the convention of the American Institute of Architects being held in Washington on the same days that your convention is being held, and which cause prevents the President of the Pittsburgh Chapter from greeting you.

You are assembled here in convention to increase the sum of knowledge by exchange and conference, and while you will be benefited by that, there is a larger compensation for you in that the result of your deliberations will undoubtedly influence both the public in general, and the officials having in charge the physical improvements in our district, and will also be of great value to the country at large.

Nature is the real cement user, and as in all other things stands as the past master in any process which has been undertaken. The cunning of man, by research in her laboratories, has accomplished much in the manufacture of cement; but the use of cement in its various forms and application has opened up a field so wide that I think very few of us who deal more or less with cement day by day have the slightest conception of what we are accomplishing by its use. The thought has come to me, that, as wheat is to the food of men, in the same relation is cement to the constructive arts today, and I believe if you will consider this simile you will see that the application is very thorough, and you will also notice this; that in the conversion of wheat into flour and the making of bread, we have turned out some very excellent bakers and some very poor bakers; in the concrete line we have turned out a splendid lot of cement users and an equally wonderful lot of cement abusers, and we look hopefully to such an organization as yours to minimize the abuse and standardize the use.

The paradoxical condition of architectural expression has always given me some pleasure and also some pain, and when we look back to the Greeks we find that in the erection of the marble temple they were influenced by the forms of wooden construction; having finished that, their sense of the artistic was not thoroughly satisfied until they had overlaid the marble with pigment and concealed the real material of the structure with an artifical or paint skin. In view of this, perhaps, we should be forgiven in our day for making houses out of wood and hollow tile and common brick, and then palming them off as cement houses by putting a half-inch of cement mortar on the

exterior surface. There are also a great many buildings being erected that are apparently brick buildings when in reality they are reinforced concrete buildings with a brick skin. Whether the day will come when we will be able to live up to architectural traditions and still frankly give to this plastic material the proper application and design I do not know, but possibly it will.

The fact remains that there seems to be a very strong distate on the part of architects for a real cement structure that will appear with a cement exterior.

Now along the line of the architects' relation to cement we are impressed with the fact that architects are often inclined to use great quantities of cement; and taking an architects specification and reading the clauses relative to the proportion of cement to be used to the coarse aggregate, you immediately (if you have any imagination) will picture that sort of thick, creaming, mass that is about the consistency of molasses, and all that sort of thing. But somehow the contractors often do not seem to see it that way; they seem to be imbued with the idea that we should keep a lot of this costly cement for the use of future ages. They are going on the conservation principle and do not wish to deprive our grandchildren of something that they should have, and that we should not deplete Nature's stock of cement, consequently the 1 : 2 : 4 mixture is more easily secured in theory than practice.

However, aside from the joke, I have no quarrel with the contractors, nor they with me, I hope. It is just as wrong to use too much cement as it is to use too little. It is a precious material regardless of its market price, and it should be spread over just as much surface as possible, and we are looking for much of the solution to your organization.

You are as an organization not operating on the "fenced in" idea of the professional man, but have his theory thrown against that of the facts and experience of the practical and commercial man in a common meeting place, so that the best of the theoretical knowledge and the best of the practical experience may be welded together into one successful, sound, good thing that can be used. We certainly hope that the results of these deliberations will be couched in such language, and with that degree of unanimity, that they will be convincing and useful to the various societies, officials, and commissions who are endeavoring to place definite values on cement and concrete in the building codes.

I thank you very much.

The following address of welcome on behalf of the Engineering Interests was delivered by James O. Handy, President of the Engineers' Society of Western Pennsylvania:

The greetings of a society of engineers to the members of a visiting society of engineers must of necessity be fraternal and are in this instance most cordial on our part. Your program shows many papers which give evidence of much painstaking and time consuming investigation and research, and the

results will be of the greatest value and interest to the profession. The Engineers' Society of Western Pennsylvania, an organization one thousand strong, offers to you the use of its rooms in the Oliver Building and the personal service of its members who can be of use to you in anything that you may wish to do in Pittsburgh, especially if you wish to visit any of the industrial works.

Pittsburgh is interesting to cement users from at least two standpoints: the first is that it is the center of cement production, being a very large producer of cement. The second point is that its use of cement is likely to increase greatly.

There are four cement factories in the neighborhood of Pittsburgh, three of them operating on limestone and shale and one on blast-furnace slag and limestone. The largest of these produces three and a half million barrels of cement a year, and is entirely operated by electrical power generated from blast-furnace gas at a point some twelve miles distant from the works. One of the works operating on limestone and shale is interesting historically, because it is the lineal descendent of one of the very first cement factories in America. In 1874 experiments began at this plant in an endeavor to make Portland cement from the limestone and blue clay there available. The efforts were necessarily very crude and the results were very uncertain. The management presently found an experienced man and in 1876 they were making such a good quality of Portland cement that they were awarded a special medal by the United States Centennial Commission.

On an occasion of this kind when there are a number of addresses of welcome, I personally, without any reflection on the very eloquent gentlemen who have preceded me or the gentlemen who will follow me, aspire to popularity in the way that a certain man Jones did. He was a candidate for public office and he had sat throughout a long evening and had listened to other speakers and finally the chairman said, "Now Mr. Jones will give his address." Mr. Jones arose smilingly and said to the audience, "My address is No. 12 Montgomery Place. Good night."

The President introduced James L. Stuart, who extended the following welcome on behalf of the Contracting Interests:

I wish to extend to you tonight a most hearty welcome to our city on behalf of the contracting interests of Pittsburgh. We are extremely glad to have you hold this convention here and we are going to take advantage of it, and gain a great deal of additional knowledge regarding the use of Portland cement and learn from those outside of our own community just what has been done along these lines. We appreciate what good these gatherings do and how much knowledge is brought together from all parts of our country by men who think and do things.

Steel and Portland cement are the two great engineering materials of the age, and the combination of these two materials has become more and more important and will continue to be of more importance. In choosing

Pittsburgh for your convention you have chosen the greatest steel center in the world, and in choosing Pennsylvania you have chosen the state which has produced the largest amount of Portland cement and the state in which the Portland cement industry of America was started. We are proud of our city and state and exceedingly glad and happy to welcome you all here to accept of our hospitality and I hope that this visit will be to your gain, as I know it will be to ours.

The President then responded:

On behalf of the members of the Association I thank Mr. O'Brien, Mr. Carstanjen, Mr. Stotz, Mr. Handy and Mr. Stuart and others who have extended a welcome to Pittsburgh. I am sure I voice the feelings of the members when I say that we will endeavor to profit by your kind invitation to see what you have here, and to avail ourselves of the opportunity for becoming better acquainted. Perhaps it may be our good fortune to leave with you some knowledge of the use of concrete, some new thoughts that may make you richer for our visit.

We thank you for your cordial welcome and we hope that you may have occasion to look back with pleasure on the Ninth Convention of the National Association of Cement Users.

The following committees of the Convention, appointed by the Executive Board, were announced by the President:

Committee on Nomination of Officers:

James O. Handy, *Chairman*, Pittsburgh, Pa.
John L. Conzelman, St. Louis, Mo.
Walter N. Denman, Springfield, Mass.
E. S. Hanson, Chicago, Ill.
W. W. Schouler, Newark, N. J.

Committee on Resolutions:

Fred K. Irvine, *Chairman*, Chicago, Ill.
Cloyd M. Chapman, New York, N. Y.
Arthur J. Maynard, State Farm, Mass.
Henry H. Quimby, Philadelphia, Pa.
R. J. Wig, Washington, D. C.

A paper on "The Use of Cement in National Irrigation Work" was read by Arthur P. Davis.

The meeting adjourned until Wednesday at 10 A. M.

WEDNESDAY, DECEMBER 11, 1912, 9 A. M.

Meeting of the Sections on Roadways, Sidewalks and Floors, and Reinforced Concrete Highway Bridges and Culverts.

President Richard L. Humphrey in the chair.

The meeting was devoted to a discussion of the aggregates and proportions for concrete to be used in the construction of roadways, sidewalks and floors, and the methods of finishing.

SECOND SESSION—WEDNESDAY, DECEMBER 11, 1912, 10 A. M.

President Richard L. Humphrey in the chair.

The report of the Committee on Reinforced Concrete Highway Bridges and Culverts was presented by the Chairman, Willis Whited, and discussed.

Mr. W. M. Denman presented a paper on "Patents Applying to Reinforced Concrete Design and Construction" which was followed by a discussion of the subject.

A paper on "The Concrete Roads of Wayne County, Michigan," was presented by Edward N. Hines, and followed by a discussion.

A paper on "Vibrolithic Concrete Pavements," by R. C. Stubbs, was in the absence of the author read by title.

The meeting then adjourned until 3 P. M.

THIRD SESSION—WEDNESDAY, DECEMBER 11, 1912, 3 P. M.

President Richard L. Humphrey in the chair.

The report of the Committee on Roadways, Sidewalks and Floors was in the absence of the Chairman, C. W. Boynton, presented by K. H. Talbot.

Revisions of the following Standard Specifications were considered and ordered to letter ballot:

Standard Specifications for Plain Concrete Floors.
Standard Specifications for Reinforced Concrete Floors.
Standard Specifications for Concrete Curb and Concrete Curb and Gutter.

Standard Specifications for Portland Cement Sidewalks.
Standard Specifications for Concrete Roads and Street Pavements.

A paper on "Effects of Electric Currents on Concrete" by E. B. Rosa, Burton McCollum and O. S. Peters was read by Burton McCollum, and was discussed.

Mr. Cloyd M. Chapman presented a paper on "Tests of Waterproofing for Concrete."

A paper was read on "Fire Test of Plastered Partitions" by L. H. Miller, followed by a discussion on "Metal Lath Construction," by H. B. McMaster.

The report of the Committee on Treatment of Concrete Surfaces was presented by the Chairman, L. C. Wason, and the proposed Standard Specifications for Portland Cement Stucco were discussed and referred back to the Committee for further report at the Convention.

The meeting adjourned until 8 p. m.

Fourth Session—Wednesday, December 11, 1912, 8 p. m.

President Richard L. Humphrey in the chair.

The annual address of the President, entitled "The Use of Concrete in Mines," was delivered by Richard L. Humphrey.

The paper on "The Use of Concrete by the Bureau of Yards and Docks, U. S. Navy," by H. R. Stanford, was in the absence of the author read by C. A. Carlson.

The following papers were then read and discussed:

"Concrete in Railroad Work," by M. A. Long.
"Application of Concrete in the Abolition of Grade Crossings," by James W. Phillips.
"Some Recent Applications of Concrete in Railroad Work," by Frederick Auryansen.

The meeting then adjourned until Thursday at 10 a. m.

FIFTH SESSION—THURSDAY, DECEMBER 12, 1912, 10 A. M.

President Richard L. Humphrey in the chair.

A paper on "Recent Applications of Concrete and Reinforced Concrete in the City of Pittsburgh," was presented by Norman S. Sprague, and discussed.

The Annual Report of the Executive Board was read by the Secretary and on motion approved.

The following motion was duly seconded and carried:

It is the sense of this meeting that it is extremely desirable that as many of the papers of the present Convention be published and distributed to the members as rapidly as conditions will permit.

Mr. E. S. Hanson for the Committee on Nomination of Officers presented the following nominations, which were accepted and the Secretary as per the unaminous instructions of the Convention cast the ballot for their election:

President, Richard L. Humphrey, Philadelphia, Pa.
Vice-President, Arthur N. Talbot, Urbana, Ill.
(To serve two years.)
Vice-President, Leonard C. Wason, Boston, Mass.
Treasurer, Henry C. Turner, New York, N. Y.
Directors (to serve two years):
First District, W. L. Church, Boston, Mass.
Second District, Edward D. Boyer, New York, N. Y.
Third District, Ernest L. Ransome, Dunellen, N. J.
Fourth District, W. P. Anderson, Cincinnati, O.
Fifth District, B. F. Affleck, Chicago, Ill.
Sixth District, Charles Derleth, Jr., Berkeley, Cal.

The time and place of the Tenth Annual Convention was left with the Board of Direction with power to act.

The meeting then adjourned to visit the plant of the H. J. Heinz Company and to meet at 3 P. M.

SIXTH SESSION—THURSDAY, DECEMBER 12, 1912, 3 P. M.

President Richard L. Humphrey in the chair.

Mr. Alfred E. Lindau presented the report of the Committee on Reinforced Concrete and Building Laws.

The report of the Committee on Measuring Concrete was presented by the Chairman, Robert A. Cummings, and the Proposed Standard Methods for the Measurment of Concrete Work submitted, was discussed and ordered to letter ballot.

The paper on "The Present Status of Unit Methods of Reinforced Concrete Construction," by John E. Conzelman, was in the absence of the author presented by Charles D. Watson.

The following papers were presented and discussed:

"Principles of Design and Results of Tests on Girderless Floor Construction of Reinforced Concrete," by T. L. Condron.

"Measurement of Actual Stresses in a Cantilever Flat Slab Reinforced Concrete Floor having Rectangular Panels," by Arthur R. Lord.

"Design of Rigid Frames in Steel and Reinforced Concrete," by Sanford E. Thompson and Edward Smulski.

"Tests to Determine Lateral Distribution of Stresses in Wide Reinforced Concrete Beams," by W. A. Slater.

The meeting adjourned to meet Friday at 10 A. M.

SEVENTH SESSION—FRIDAY, DECEMBER 13, 1912, 10 A. M.

President Richard L. Humphrey in the chair.

The proposed Standard Specifications for Portland Cement Stucco in revised form were presented by C. M. Chapman, and were ordered to letter ballot.

The following motion was adopted:

Resolved, That the Committee on Treatment of Concrete Surfaces be instructed to make an investigation of the durability of the various types of metal lath and wood lath used for Stucco and to report at the next Convention.

The report of the Committee on Standard Specifications and Methods of Tests for Concrete Materials was presented by Sanford E. Thompson, Chairman. On motion, this report was approved.

Mr. William M. Kinney, Chairman, presented the report of the Committee on Aggregates.

The following papers were then read and discussed:

"The Constitution of Portland Cement. Some Results Obtained at the Experimental Cement Plant of the Bureau of Standards," by P. H. Bates.

"Sand and Gravel Washing Plants," by Raymond W. Dull.

The meeting adjourned until 3 P. M.

EIGHTH SESSION—FRIDAY, DECEMBER 13, 1912, 3 P. M.

President Richard L. Humphrey in the chair.

The following papers were read and discussed:

"The Manufacture of Concrete Products in Germany," by Julius Carstanjen, Jr.

"The Coloring and Texture of Concrete Products," by Adolph Schilling.

"The Progress of the Work of the Bureau of Standards in Cement and Concrete," by Rudolph J. Wig.

The paper on "Reinforced Concrete Silos and Small Grain Bins," by E. S. Fowler, was read by title.

The Committee on Resolutions, Fred K. Irvine, Chairman, then presented the following resolutions, which were unanimously adopted:

Resolved, That this Association in convention assembled acknowledge the receipt of, and express its thorough appreciation for, the most hearty greetings and best wishes for a successful convention extended to it by the Deutscher Beton-Verein, and be it

Further Resolved, That the Secretary transmit the above resolution with the best wishes of this Association for the success of the Deutscher Beton-Verein.

WHEREAS, The work of this Association and the field of its activity has been greatly enlarged during the past year by dues from contributing members, and by donations from individuals and corporations interested in the work, therefore, be it

Resolved, That the Association hereby express its thanks to those who have thus generously aided in the work, and that the Secretary be directed to send a copy of this resolution with a personal letter to each contributing member, and to each individual firm or corporation from whom donations have been received.

Resolved, That the thanks of this Association are hereby tendered the officials of Pittsburgh and the representatives of the local engineering and concrete interests for their hearty welcome, to the press and citizens of Pittsburgh for their co-operation in making this, the Ninth Annual Convention, a notable success, and to the guests of the Association for their assistance in this success by the contribution of their interesting and valuable papers.

Resolved, That the thanks of this Association are hereby tendered to the members who have aided by the presentation of papers, to the several committees whose efforts have added this meeting to the long series of successful conventions, to the technical press whose recognition and assistance in the work of this organization is gratefully acknowledged, and to its various officers but particularly to its retiring Treasurer, Mr. H. C. Turner, and to its President, Mr. Richard L. Humphrey, for their untiring devotion to the interest and welfare of this Association.

Resolved, That the proceedings of this Convention of the National Association of Cement Users shall be published in a monthly journal to be known as the Journal of the National Association of Cement Users, provided arrangement can be made with a publishing company whereby the cost shall not exceed $2.00 per member per year for the entire proceedings.

Believing that such an arrangement is possible it is the sense of this meeting that the Board of Direction of this Association accept a proposition similar to that suggested by a member of this Association, and the Board of Direction is hereby instructed to take immediate action to make this resolution effective.

The President thereupon declared the meeting adjourned, *sine die.*

National Association of Cement Users.

PROCEEDINGS

OF THE

NINTH CONVENTION

This Association is not responsible, as a body, for the statements and opinions advanced in its publications.

THE USE OF CONCRETE IN MINES.*

Annual Address by the President,

Richard L. Humphrey.†

The use of concrete in place of timber in mines dates back a number of years, but it is only within recent years, coincident with the development of the use of reinforced concrete and consequent economies in construction, that it has come into more general use, especially in Europe where the policy of building permanent works in connection with mining operations is in sharp contrast with the general practice in this country of building temporary structures.

During trips to Europe the speaker has given this matter considerable study and has visited the principal mines in England and the Continent with a view to preparing a report for the United States Bureau of Mines, and it is with the permission of the Director of that Bureau that this paper is presented.

During these visits to Europe the inspections were of works connected with the mining of coal. The most striking feature of the collieries in Continental Europe is the permanent character of

* Address delivered with permission of the Director, U. S. Bureau of Mines.

† Consulting Engineer, Philadelphia, Pa.

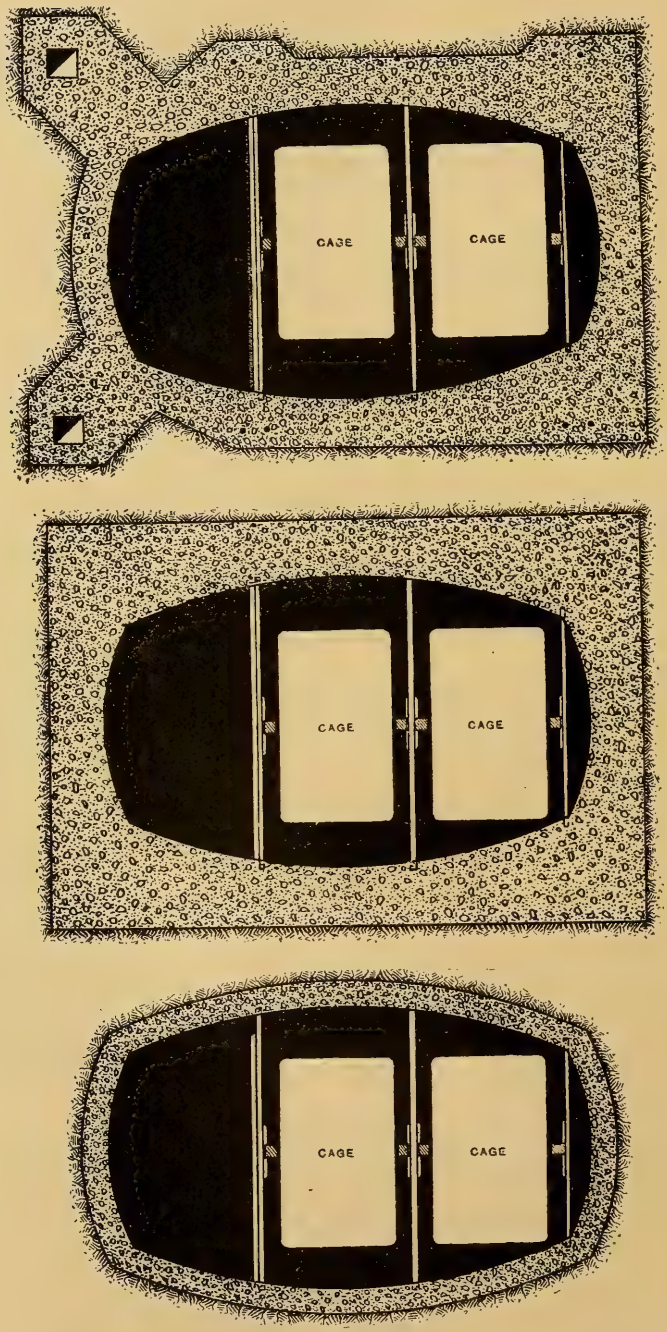


FIG. 1.—SECTIONS OF CONCRETE SHAFT LININGS AT BRIDGEPORT COLLIERIES, PENNSYLVANIA.

the head works, shafts, entry chambers and galleries. In the care of the miners, much better facilities are provided than is the case in this country or even in England. In most cases governmental regulations make this compulsory. When one visits a German mine it is almost a shock to see the miners coming from the mines with clean faces and clothes instead of the unkempt appearance which characterizes the miners in America as well as those in England. The regulations prescribe a change of clothing on entering the mine; and on leaving, after a bath, the miner replaces his mine with his street clothes. It is a revelation to an American to see miners free of coal dust and soot, leaving the mine clean and neat and of respectable appearance. The system that makes cleanliness a necessity also gives careful consideration to the structures entering into mining operations. The shafts both in England and in Continental Countries are almost always permanent in character and are generally of concrete. When you consider that in some parts of Germany the shaft is some three thousand feet deep, you can readily understand that in order to operate such mines a lining of a permanent character is necessary. These linings are installed in various ways; the shafts are either circular, elliptical or nearly rectangular in section. A common method is to build the usual forms and deposit concrete between the form and the walls of the shaft. The illustration, Fig. 1, is of shaft linings used in the Bridgeport collieries in Pennsylvania.

In Europe it is the more common practice to use the arrangement shown in Fig. 2, which consists of a movable form with a work platform having an opening in the latter through which the material is removed; as the form sinks into the mine, sections of the ring are concreted as shown, the method being somewhat similar to the construction of a chimney excepting that the concrete is deposited downwards instead of upwards. In very deep shafts it is the practice to place at intervals a cast iron ring which is forced rigidly against the walls of the shaft and serves as an additional support for the lining. It is the speaker's opinion that this is an outgrowth of masonry linings (stone or brick), and the practice is not changed when concrete is substituted. This cast iron ring is placed in sections in the shaft, fastened and grouted solidly to the wall. Where there is much water, provision is generally made to drain it as the construction proceeds.

In the method shown in Fig. 2, the materials are generally lowered to the platform and the concrete is mixed by hand or with a small mixer and deposited from this platform.

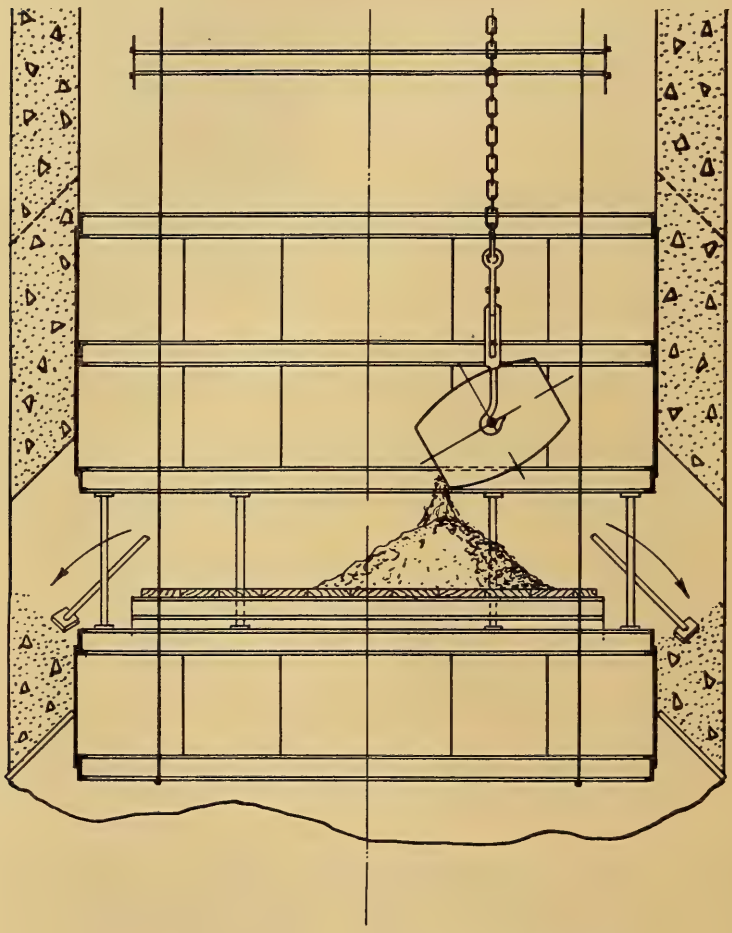


FIG. 2.—APPARATUS FOR LINING SHAFTS WITH CONCRETE.

At Gelsenkirchen, Germany, this method was used for lining the shaft which was 6 meters (about 20 ft.) in diameter; the form was moved about every 3 or 4 days. Another method of lining consists in molding the ring in segments (Fig. 3), allowing these

to harden and then placing them in position in about the same manner that the cast iron segments previously described are placed. These blocks or segments of rings have grooves around the edges and in these grooves (both horizontal and vertical) the reinforcement is placed; when a section of lining is thus completed the space between the ring and the wall of the shaft is solidly filled with grout which is introduced through a hole in the center of the block. Where this method of lining has been used it has proved very

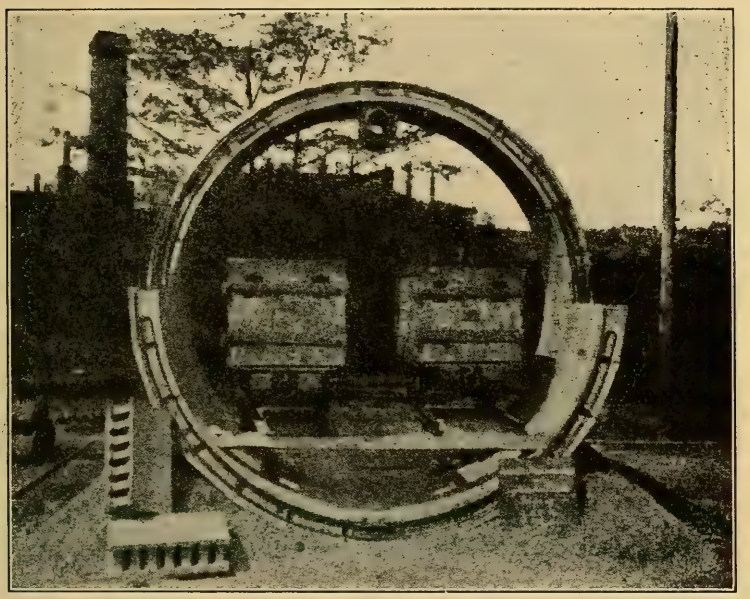


FIG. 3.—SEGMENTAL BLOCKS OF CONCRETE FOR LINING SHAFTS AND GALLERIES.

economical. Such a method was used in the main shaft of the Thurcroft colliery in England. A shaft about 650 meters (2,133 ft.) deep lined in this manner was inspected at Radbod, Germany; the segments were about 12 in. thick and were grooved to receive the ½-in. diameter reinforcement.

One of the most interesting mines was that at Marles, in the Pas de Calais District in France, where a shaft which had been in use for some time had caved in. The shaft was abandoned and remained in disuse for a number of years. When the speaker

visited this mine they had begun to excavate the material including the head works which had caved in leaving a very large hole in the ground. The method of procedure was to build first a reinforced concrete frame 28 meters (about 92 ft.) square on top of the ground spanning this hole, attached to which were long rods for supporting the concrete lining of the new shaft which is about 18 ft. in diameter. The soil was intermixed with quicksand and saturated with water and was quite difficult to handle. The cementation process was resorted to, to stiffen this soil sufficiently to enable them to sink the concrete lined shaft. As the excavation proceeded the form suspended by the reinforced concrete frame was lowered and then horizontal holes were drilled leading from the shaft into the soil. These extended for a considerable distance into the soil and into these holes cement grout was forced under pressure; after an interval of time, to permit the soil intermixed with the cement to harden, the earth was excavated and the form was lowered; about 575 ft. of the lining had been constructed in this manner at the time of the speaker's visit and he understood that the shaft had to go at least 1,300 ft. before the coal was reached.

In the cementation process radial holes are drilled in the ground in which the shaft is being sunk for the introduction of the cement grout; Fig. 4 gives an idea of how the grout is forced through these holes into the soil to be cemented. It is frequently necessary to use very high pressure and experience shows that the greater number of holes drilled, the more effective is the process. The rate of hardening of the cement also forms an important item in the successful use of this process. The process has also been successfully used for increasing the bearing power of the soil for the foundations of structures.

At the bottom of the shafts there is the main entry gallery and frequently there are auxiliary entry galleries between this and the surface. The main gallery is almost universally of a permanent character, built chiefly of masonry; although there is a decided tendency to use concrete and one may see many galleries in which it has been used.

The galleries or mine entries differ in design, which is determined by local conditions. In order to secure maximum overhead clearance steel I beams are frequently used in the roof and the space between them filled with concrete; the side walls are of brick,

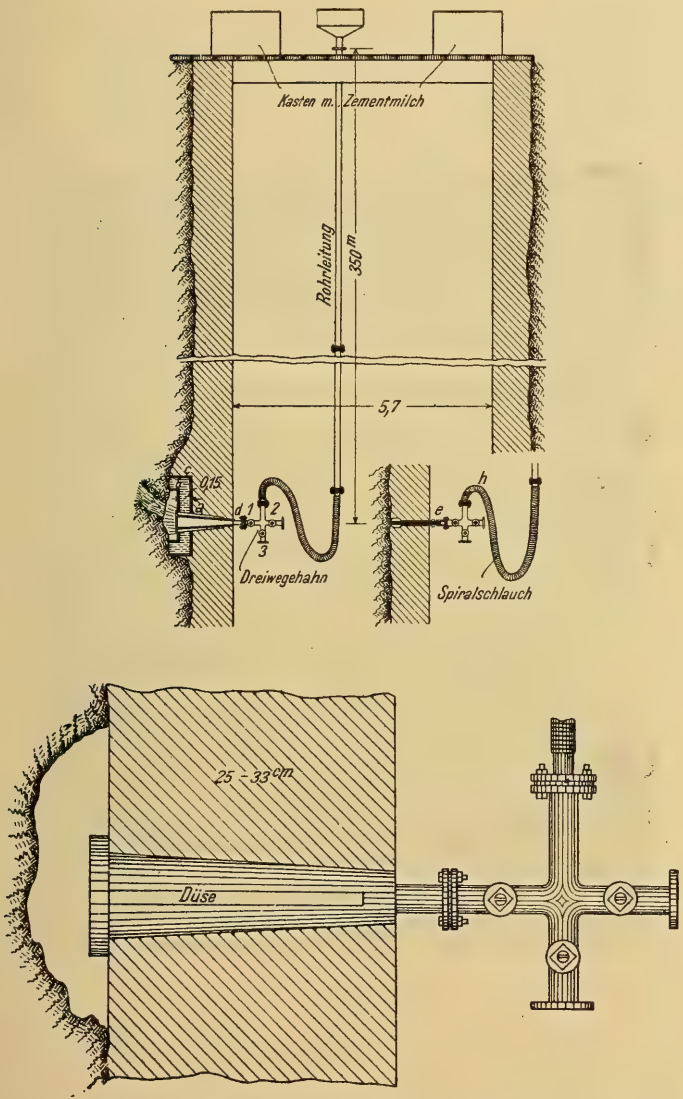


FIG. 4.—METHOD OF GROUTING IN THE LINING OF SHAFTS OR IN THE CEMENTATION PROCESS.

stone or concrete. Another method is to form an arch of corrugated metal, and fill the space between this and the roof, including the side walls, with concrete. Still another method shown in Fig. 5 is to form the walls and roof of concrete, which is the most satisfactory method. In some cases, where the mine is several thousand

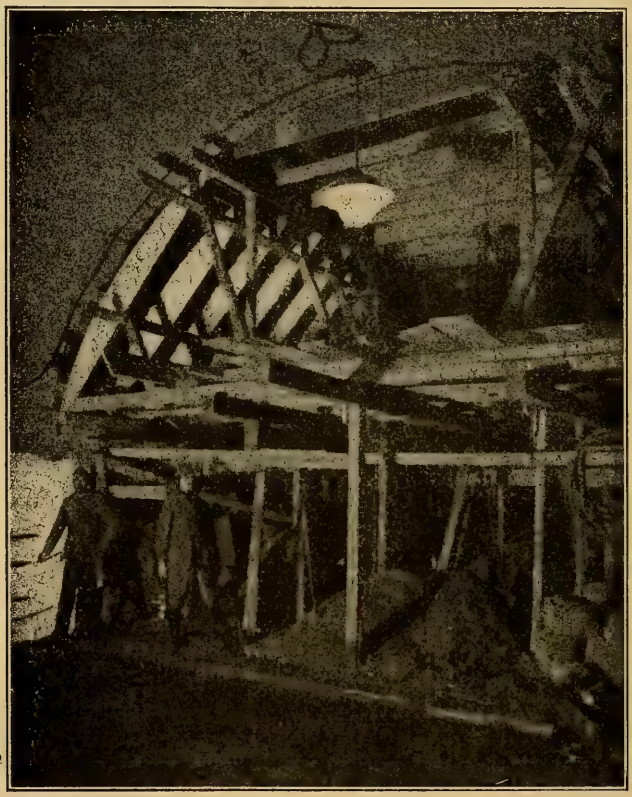


FIG. 5.—METHOD OF LINING WALLS AND ROOF WITH CONCRETE.

feet below the surface, the enormous pressure at this great depth necessitates unusual types of construction; it is found that where the floor of the gallery is not particularly hard, it constantly rises and the clearance in the gallery must be maintained by constant excavation. In such cases the type of construction shown in Fig. 6 is resorted to not only for the main but for the branch galleries also.

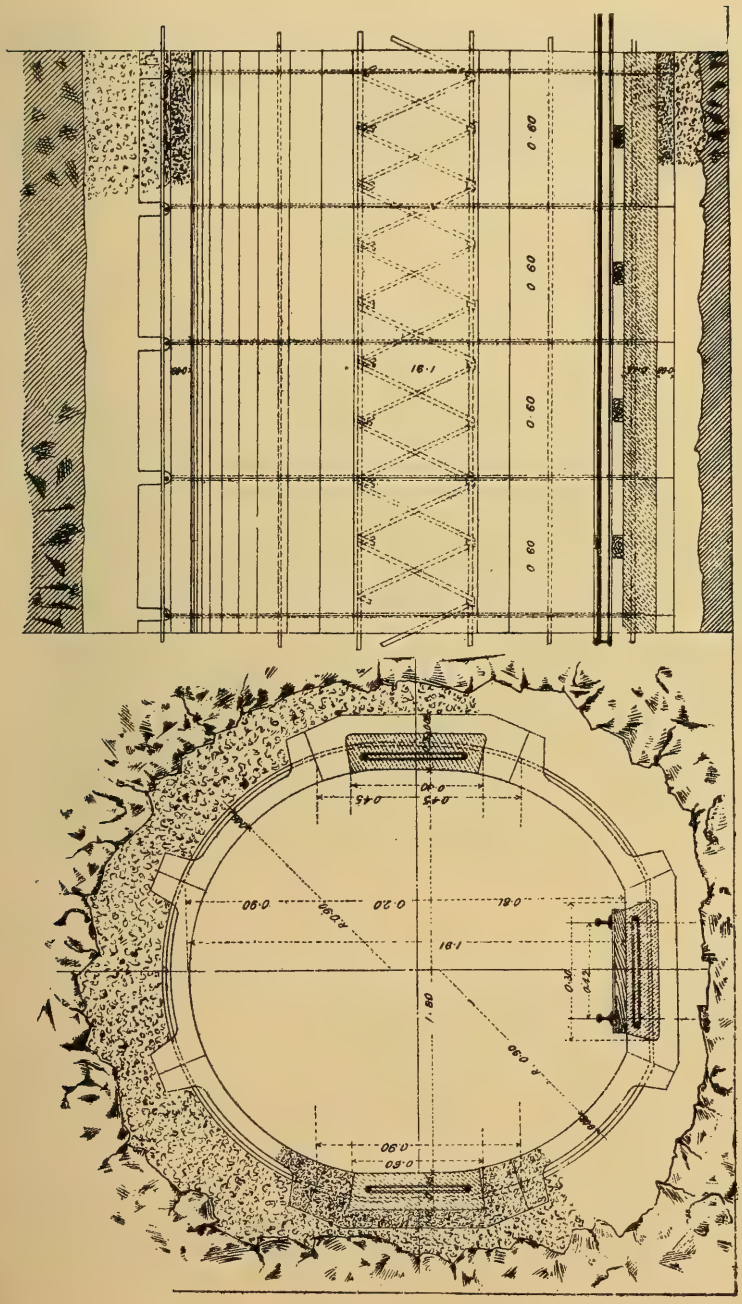


FIG. 6.—METHOD OF LINING GALLERIES WITH REINFORCED CONCRETE, AT MACHIENNE COLLIERY, CHARLEROI, BELGIUM.

This application of reinforced concrete is in the Machienne Colliery, Charleroi, Belgium; this colliery is said to be the deepest in the world (about 3,900 ft.) and the pressure on the lining of the gallery is very great. It is highly probable that no more economical or efficient material than concrete could be used under these conditions.

Fig. 7 is of interest because it illustrates a modern trend in shaft and gallery linings; this is a view of a cement products yard at Hibbing, Minnesota, for casting the separately molded

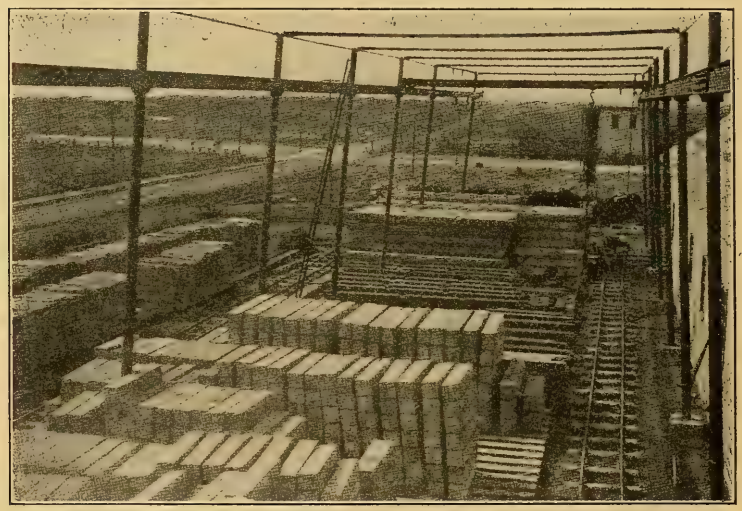


FIG. 7.—CEMENT PRODUCTS PLANT, FOR MAKING CONCRETE MEMBERS FOR LINING MINE SHAFTS, AT HIBBING, MINNESOTA.

members for such linings. In Fig. 8 may be seen an inclined shaft lining constructed of separately molded concrete members by the Ahmeek Mining Company, Michigan.

In several mines visited in Germany the circular gallery was in use, built after the manner of reinforced concrete pipe. This type is necessary in deep mines where the pressure is very great or where roof and floor are very bad.

Where coal is actually mined, the problem becomes more difficult; in the deeper mines in Germany it is impossible to find any type of prop which will withstand these enormous pressures.

The speaker frequently saw steel props buckled up like pieces of wire and wooden props crushed in the manner shown in Fig. 9. Under such conditions ordinary timber is insufficient and steel has not much greater efficiency. Resort has been had to reinforced concrete props with considerable success; although by reason of the weight of these props they have proven somewhat difficult to handle; if the floor of the mine is yielding there is a tendency for the prop of any material to punch through, but less so in the case of the reinforced concrete prop. This permits settlement of the roof and necessitates constant excavation in order to maintain the clearance necessary for the movement of the mine cars. Where

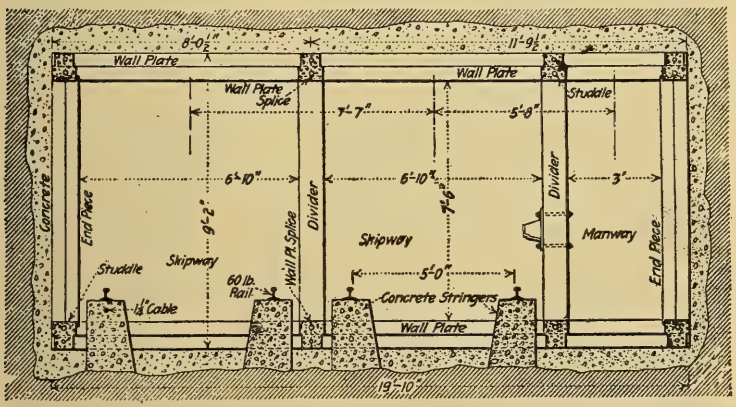


FIG. 8.—INCLINED SHAFT LINING IN WHICH SEPARATELY MOLDED MEMBERS OF CONCRETE WERE USED, AHMEEK MINING COMPANY, MICHIGAN.

reinforced concrete props have been used they have been found to be economical and durable.

In many cases, however, the construction of a reinforced concrete ring has been resorted to as the most economical method. Where the floor of the mine is of such poor quality that it is forced up under pressure, the props sink into it and it is almost impossible to secure a firm footing for the props. In some instances where the roof is good and the floor bad, it is only necessary to form an arched bottom in order to successfully meet the difficulty. The speaker recalls one mine in Germany at Dortmund where a horseshoe-shaped gallery lining of concrete, not particularly

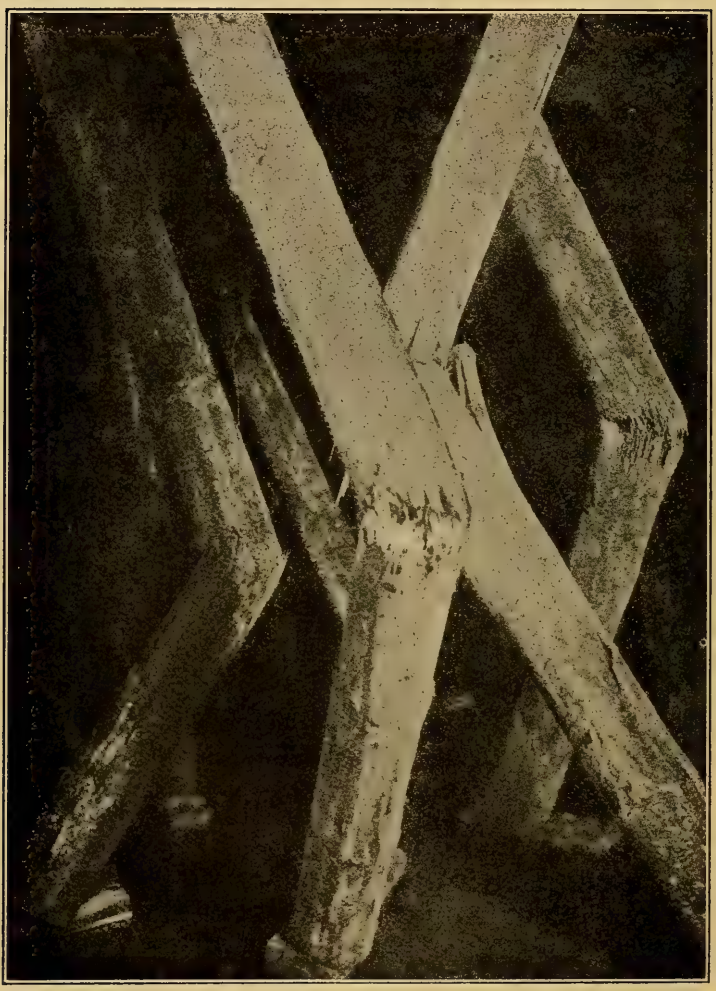


FIG. 9.—MANNER OF FAILURE OF WOODEN MINE PROPS.

well reinforced, had been badly crushed under such conditions, and in extensions to this gallery a circular reinforced concrete lining was used.

One frequently finds underground stables and similar structures built of reinforced concrete, which is an admirable material for any underground structure because it offers the greatest resistance to fire of any of the available materials of construction.

At the top of the shaft is located the machinery for raising and lowering the mine cars. These structures have usually been built

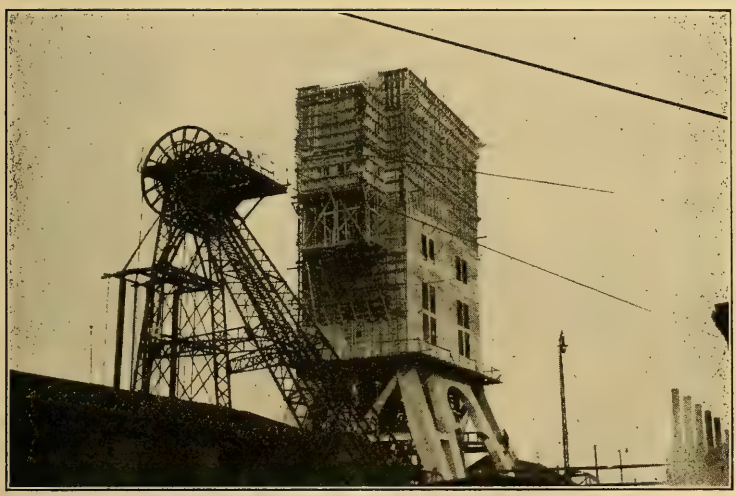


FIG. 10.—REINFORCED CONCRETE HEAD HOUSE AT CAMPHAUSEN, GERMANY.

of structural steel, but because of rigidity, durability and economy in cost of construction and maintenance, reinforced concrete is coming into general use and is proving eminently satisfactory.

Fig. 10 is such a structure erected at Camphausen, Germany; in this structure an electric motor-driven drum, carrying the cable by which the mine cars are raised and lowered in the shaft, is located in the upper part, the lower part being completely free of obstruction. The speaker was informed that the cost of this structure was materially less than steel and that it has proved thoroughly satisfactory. The structure shown in Fig. 11 is

located at the colliery at Heapstead, England; in this picture may be seen a frame of reinforced concrete carrying the head house and a reinforced concrete stairway leading to one of the buildings.

A disastrous fire, destroying the coal breaker and other mine top structures will frequently shut down the mine for considerable time. When the lost profit on business thus interrupted is taken into consideration, the cost of reinforced concrete construction proves very much less expensive than the cost of any other material

FIG. 11.—REINFORCED CONCRETE USED IN COLLIERY HEAD WORKS, HEAPSTEAD, ENGLAND.

because of its high fire resistance; such construction is frequently very little if any more expensive than the usual timber structure.

A most interesting example of this use of reinforced concrete is the breaker house shown in Fig. 12 erected by the Delaware, Lackawanna and Western Railroad, to replace the timber breaker destroyed by fire known as the Taylor Coal Breaker,* which the speaker is informed cost little more than that of a breaker of wood.

Fig. 13 is a coal storage bin erected in Germany, and illustrates a very common use of reinforced concrete.

* *Proceedings*, Vol. VII, p. 371.

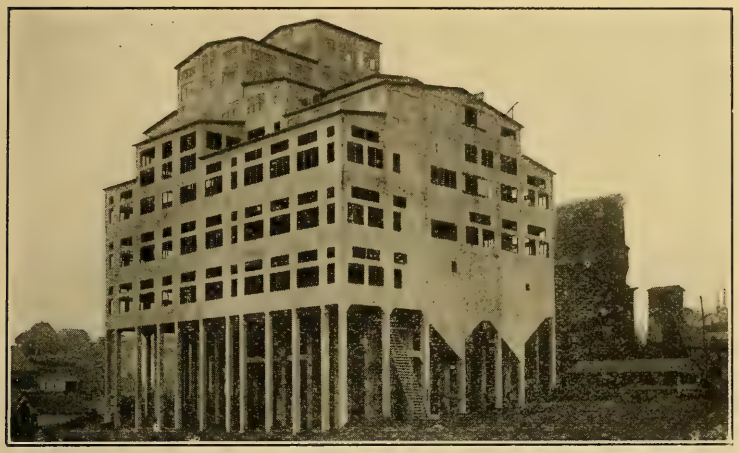


FIG. 12.—REINFORCED CONCRETE COAL BREAKER OF DELAWARE, LACKAWANNA AND WESTERN RAILROAD.

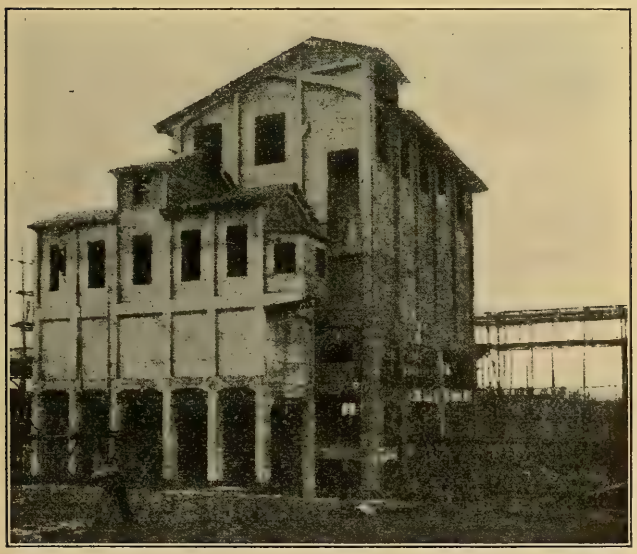


FIG. 13.—COAL STORAGE BIN AT COLLIERY IN GERMANY.

The use of reinforced concrete on the Continent for the construction of wash rooms for the miners, for engine buildings and

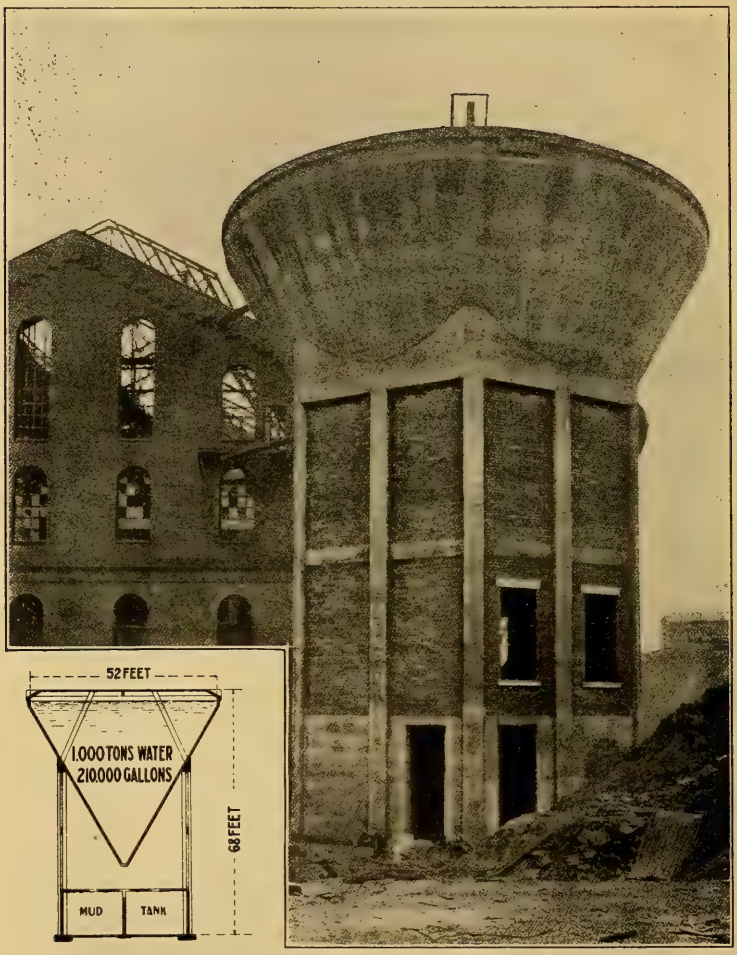


FIG. 14.—REINFORCED CONCRETE SETTLING TANK, SILVERWOOD COLLIERIES, ROTHERHAM, ENGLAND.

other structures, is very general and some very pleasing ornamental buildings are to be seen. These structures are kept in most excellent condition and there is not the appearance of dilapidation so

commonly seen about the mines in this country; a further commendable practice is the removal of rubbish, so that the vicinity of the mine presents an orderly, business-like appearance.

The washery shown in Fig. 14 is that used in connection with the Silverwood Collieries, Rotherham, England. A number of similar tanks have been successfully installed at other collieries. The water from the washery, heavily charged with coal dust, is pumped into the tank and settles and the clear water is returned to the washery. The coal dust removed is used with the coal that is briquetted.

A structure of extraordinary interest to the speaker was that which is shown in Fig. 15 (Plate I); this is an articulated structure of reinforced concrete built on the unit system; it is located at the mine at Floreffe, Belgium. At the left may be seen the entrance to the mine and at the right the coal tipple. This structure was built of separately molded members, which were allowed to harden and were then assembled. The arch spanning the brook is three hinged and the manner of testing it with one of the test loads on it may be seen in the picture. The structure was erected with extraordinary speed and cost a great deal less than would a structure of any other material.

Another structure that was most interesting was the explosive test gallery of reinforced concrete built at Lievin near Lens in the Pas de Calais district. This gallery is 65 meters long, 1.85 meters high and 1.50 meters wide. The use of concrete in connection with a similar gallery at the U. S. Bureau of Mines is also worthy of note.

It is evident, the speaker thinks, that the use of concrete in mines will in the future be much more rapid and extensive than in the past; the use of concrete members separately molded as a substitute for timber, and segmental linings of concrete for shafts, tunnels and galleries, will meet with rapidly increasing favor.

The permanence of concrete construction, its low first cost and cost of maintenance, in addition to its plastic properties by which it may be readily formed into structures designed to satisfactorily resist the stresses covering widely varying conditions of use, are all elements which render concrete one of the most desirable materials for use in mine operations. The speaker is of the

opinion that as the mine operator uses concrete and reinforced concrete he will acquire greater confidence and with this confidence will come a much greater application. It is certainly a most promising field for exploitation and it is hoped that the Association may be a medium by which the use of this material for this purpose may be greatly increased.

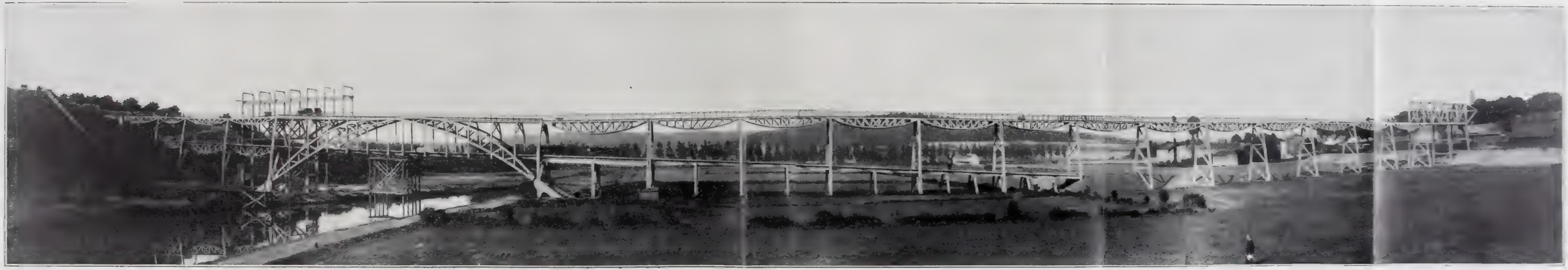


FIG. 15.—REINFORCED CONCRETE BRIDGE, TRESTLE AND COAL TIPPLE, AT FLOREFFE, BELGIUM.

EFFECTS OF ELECTRIC CURRENTS ON CONCRETE.

By E. B. Rosa,* Burton McCollum† and O. S. Peters.‡

I. Introduction.

During the last few years attention has been called to the possibility of damage to reinforced concrete structures by stray currents from electric railways and other power sources. The laboratory experiments of Toch, Knudson, and Langsdorf in 1906 and 1907 showed quite clearly that under certain circumstances the passage of electric currents from the reinforcing material out into the concrete gave rise not only to serious corrosion of the reinforcing material but also to cracking and disintegration of the surrounding concrete. Since then, numerous laboratory experiments have been carried out by various investigators, all tending to confirm the earlier observations in regard to the destruction of the concrete, but giving rise to numerous conflicting theories as to the cause of the phenomena observed. Following the early demonstrations of the possibility of damage to concrete by electric currents, reports of serious damage to certain concrete buildings, bridges, etc., became current and considerable apprehension has been aroused in some quarters that great damage may be in progress due to this cause. The subject was brought directly before the Bureau of Standards by numerous letters of inquiry from engineers, contractors and corporations requesting information in regard to the probable extent of the damage and the most feasible methods of preventing it. Although a good deal of work had been done showing that under certain conditions, readily producible in the laboratory, blocks of reinforced concrete could be completely destroyed by electric currents, there remained a wide diversity of opinion as to the cause of the phenomena observed, and no work at all had been published tending to show to what extent and under what circumstances damage might be expected under practical conditions, or how

*Assistant Director, Bureau of Standards Washington. D. C.
† Asssociate Physicist, Bureau of Standards, Washington, D. C.
‡ Assistant Physicist, Bureau of Standards, Washington, D. C.

trouble from this source might best be prevented. Recognizing the great practical importance of the subject and acting in response to requests from numerous sources, the Bureau of Standards has undertaken a thorough investigation of the cause and nature of the phenomena observed in concrete under the influence of electric currents, the extent to which damage has occurred or is likely to occur in practice from this cause and the best methods of mitigating the trouble under practical conditions. The work was begun during the summer of 1910 and certain phases of the investigation are still in progress, but enough work has been completed to justify the publication of a report of progress at this time. The investigation has comprised three general branches, as follows: (1) Laboratory investigations relating to the cause and nature of the phenomena caused by the passage of electric currents through concrete. (2) Investigations in the field with the view of establishing definitely the probable extent of the danger in practice and the circumstances under which trouble is most likely to occur. (3) A study of the various possible means of mitigating trouble from this source, leading to specific recommendations based thereon.

II. Investigations Relating to the Nature and Cause of the Phenomena Resulting from the Passage of Electric Currents through Reinforced Concrete.

1. Form and Composition of Test Pieces.

At the outset of the investigation a number of the experiments described by previous experimenters were repeated in order to verify their results and to afford an opportunity for studying at first hand the phenomena previously observed. At the same time numerous modifications of these experiments as well as many radically different ones, designed to throw special light on particular phases of the subject, were instituted. In this preliminary work one general type of test piece was adhered to as far as practicable. A number of well known brands of Portland cement, all of which conform to the requirements of the Standard Specifications for Portland Cement, were selected, together with sand and stone similar to those used in large quan-

tities by builders in the city of Washington for reinforced concrete construction. Using filtered Potomac River water, these ingredients were made into a $1 : 2\frac{1}{2} : 4$ concrete; a proportioning which gave a very dense mass. In many special experiments, especially where chemical analyses were to be made, quartz sand and distilled water were used as noted later. The test pieces were molded as cylinders 6 in. in diameter and 8 in. long with an

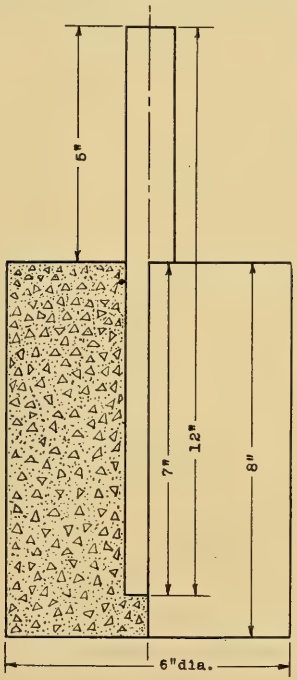


FIG. 1.—GENERAL TYPE OF TEST PIECE.

electrode embedded on the axis of each cylinder as shown in Fig. 1.

The test pieces were allowed to remain in the molds until set sufficiently for handling and they were then removed and buried in wet sand for approximately 20 days. At the end of this time they were taken out of the sand and placed in the shelves of a case where they were wet down occasionally until used.

Where iron served as material for the embedded electrode it was cleaned of scale and rust by pickling in dilute sulphuric acid. After the pickling process was completed the iron was removed from the acid, dipped in limewater and after being numbered and weighed immediately embedded in the test piece. This insured the entire absence of rust to begin with. Electrodes other than iron were sandpapered or otherwise cleaned to a bright surface before embedding. In the report of each experiment the type of electrode used is given as well as any departure from the procedure described above.

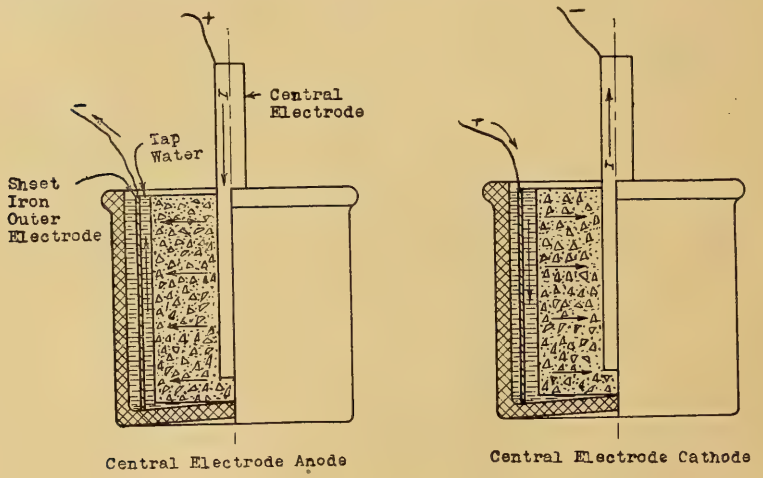


FIG. 2.—GENERAL ARRANGEMENT OF TEST PIECES.

2. ARRANGEMENT OF TEST PIECES IN VESSELS.

Each test piece was placed in an earthenware jar $8\frac{1}{2}$ in. in diameter and 9 in. deep. For an outer electrode an 8 x 24 in piece of No. 24 gauge sheet iron was rolled into a cylinder 7 in in diameter by 8 in. long and placed around the test piece. The arrangement is shown in Fig. 2. The jar, with the exception of cases otherwise designated, was filled to about 1 in. from the top of the test piece with tap water. Rubber covered wire leads were soldered to the outer sheet iron electrode and to the project ing end of the embedded metal. By connecting these leads to the proper poles of the electric circuits the embedded metal could be made either anode or cathode.

The description of the phenomena observed is given under two heads, viz: (1) Anode effects, or those effects observed where the current flows from the embedded iron out into the concrete, and (2) Cathode effects, or those occurring when the current flows from the concrete to the embedded iron. In both cases experiments were carried out under both high and low voltage and as the results were radically different in the two cases, they are discussed below under separate heads. In the high voltage tests the test pieces were subjected to voltages varying from about 50 to 70 volts and the current flow was continuous except for infrequent and irregular intervals when work was being done on the test pieces themselves or on the electric circuits to which they were connected. The time during which they were disconnected amounted to less than 1 per cent. of the total time. In the low voltage tests the current was maintained for $7\frac{1}{2}$ hours each working day for the first 8 weeks, after which the current was on for 24 hours each day with the exception of Sundays and holidays, and an occasional short interval when the machine was shut down for repairs. In calculating the number of hours of a test only the time during which current flowed was considered.

3. ANODE TESTS ON HIGH VOLTAGE.

Only a comparatively small number of test pieces were subjected to high voltage because such conditions are abnormal and would rarely, if ever, be encountered in practice. A few of these were made, however, to check the observations of previous experimenters and the results noted in these tests are recorded briefly below. In the preliminary tests on high voltage, in which iron served as the anode, 11 test pieces were used. They were all of like proportions, viz: $1 : 2\frac{1}{2} : 4$ and were connected two in series on 115-v. direct current. Condensed data on these tests are give in Table I. All of the test pieces behaved in substantially the same manner. During the first few hours there was an appreciable rise in the external temperature. In most cases this rise amounted to from 12 to 25 deg. C. above room temperature as measured by a thermometer placed within the electrode, if hollow, or against the electrode at the point of emergence from the concrete and covered with cotton waste if the electrode was solid. This rise of temperature usually reached its maximum

TABLE I.—HIGH VOLTAGE TESTS ON REINFORCED CONCRETE WITH IRON ELECTRODES.

Test Piece No.	Voltage		Average Current to Cracking amperes.	Total Ampere Hours.	Ampere Hour Density, per sq. in.	Resistance in ohms.			Hours to Cracking.	Hours of Test.	Brand of Cement used.	Electrode used.	Age of Concrete at Test.
	at Beginning of Test.	at Cracking				at Beginning of Test.	at Cracking	at End of Test.					
1	55.0	38.0	0.75	17.25	1.04	64	90	1,766	23	1,128	A	¾-in. round iron	40 days
2	59.0	78.0	0.75	17.25	1.04	70	171	2,100	23	1,128	"	"	"
3	59.5	63.0	0.57	13.20	0.86	79	157	2,330	23	1,128	"	"	"
4	54.5	53.0	0.57	13.20	0.86	72	132	1,530	23	1,128	"	"	"
5	56.5	77.0	0.74	17.02	0.63	66	140	2,700	23	1,200	"	1-in. pipe	56 days
6	55.5	69.5	0.79	18.10	0.67	69	124	2,400	23	1,200	"	"	"
7	55.0	53.5	0.57	13.00	0.48	58	119	1,920	23	1,200	"	"	"
8	62.0	70.0	0.74	17.02	0.63	71	132	2,810	23	1,200	"	"	"
9	24.5	54.0	0.14	13.80	0.83	122	275	5,250	96	3,068	B	¾-in. round iron	"
10	58.0	57.0	0.42	30.96	1.08	100	140	140	72	72	C	1-in. pipe	9 mos.
11	58.0	57.0	0.39	28.20	1.00	100	163	163	72	72	"	"	"

and diminished very considerably before any damage to the concrete was apparent. After a few hours of current flow bubbles of water containing iron rust began to appear around the anode and cracks soon developed in the concrete. The manner in which the cracking occurred was the same in all test pieces. First, there appeared a very fine crack beginning at the iron and extending outward in practically a radial direction. This crack soon extended to the outer surface of the concrete cylinder and then gradually widened, while at the same time two or three other and smaller cracks would appear on the opposite side of the anode from the initial crack and also radiated from the center.

The manner in which these cracks occur and develop is strongly suggestive of a wedge-like action slowly applied at the center of the test piece. After the first crack has extended from the center to the outer surface of the concrete, the test piece can readily be pried open with a screwdriver and separated into several pieces. The individual pieces of concrete thus obtained are, however, to all appearances as strong as similar specimens of concrete that have not been subjected to the influence of electric currents, and are broken with a hammer with the same difficulty. There is no indication whatever that the cement decayed or deteriorated in any way as reported by some earlier investigators, but on the contrary the appearance of the concrete and its mechanical properties, all indicate very strongly that there has been no such action. This is fully substantiated by special experiments described in a later section which appear to show quite conclusively that in the body of the concrete, remote from either electrode, the current has no apparent effect on the concrete.

As shown in Table I most of the test pieces cracked in about the same time, the first 8, composed of Brand A cement, having shown cracks at the end of 23 hours. Test pieces 10 and 11, containing Brand C cement, required 72 hours while 9, made with Brand B White Portland cement, required 96 hours to develop a fracture. In the latter case, however, it will be noted that the voltage at starting was only 24.5 volts, this gradually rising to 54 volts at the time of cracking. This low voltage at the start was caused by the placing of a rheostat in series with the test piece in order to prevent an unduly rapid rise of temperature.

As the test piece warmed up the resistance of the rheostat was cut out by steps until full voltage was on.

The variation of the electrical resistance of the test piece with time is of particular interest. In Table I these resistances are given at three stages, viz: at starting, at the time of cracking and at the conclusion of the test. It will be noted that the first 8 test pieces, containing Brand A cement, show a fair degree of uniformity in initial resistance, the average of the 8 being 68 ohms. At the end of 23 hours, when cracks first appeared this

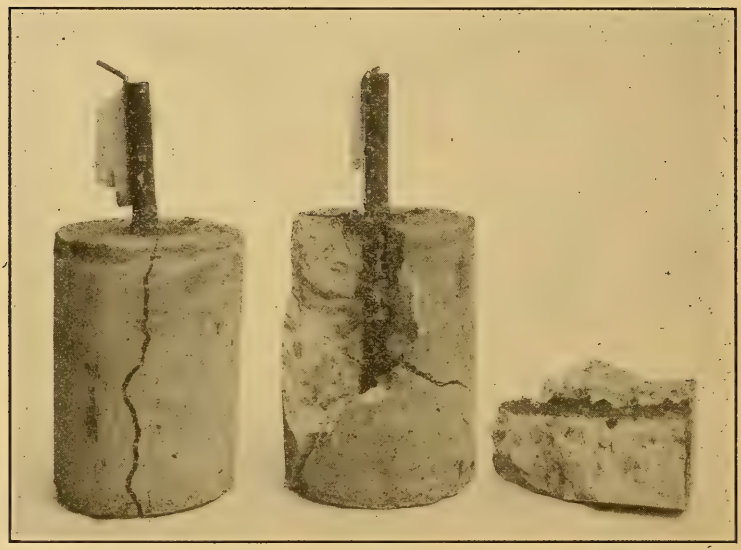


FIG. 3.—EFFECT OF ELECTRIC CURRENT, HIGH VOLTAGE, ON TEST PIECES.

had increased somewhat, the average then was 133 ohms. At the end of the test this average had risen to 2,194 ohms, or over thirty-two times its initial value. This enormous rise in resistance with time is of the greatest importance, particularly from the practical standpoint, and is referred to again in another part of this paper.

On breaking open the test pieces the embedded iron was found to have been very badly corroded, thick layers of scale consisting largely of the oxides of iron having formed all over the surface and to some extent in the voids in the concrete adjacent

to the iron. In the test pieces left in circuit for a long period after cracks developed, the deposit of oxides spread for some distance out into the cracks, but in test pieces 10 and 11 which were removed and opened immediately after cracking this was not the case. Upon breaking open the solid portions of the concrete, it was found that the discoloration due to the products of corrosion had not penetrated into the body of the concrete more than an eighth of an inch or so.

The general appearance of the test pieces is shown in Fig. 3 which is a photograph of two test pieces, one exhibiting the initial crack as caused by the passage of the current and the other showing the appearance of a similar test piece after having been pried open to expose the embedded iron. They form very striking examples of what may happen to reinforced concrete when subjected to comparatively high voltages. They should not, however, be assumed to represent a condition that is liable to be of frequent occurrence in practice, as will presently appear.

4. ANODE TESTS ON LOW VOLTAGE.

A much greater number of test pieces were subjected to 15 volts and less than to a high voltage. At the outset of the investigation 90 test pieces containing iron electrodes were placed in circuit and watched for a period of about 5,500 hours ($7\frac{1}{2}$ months), during which time careful records were kept of voltage and current flow. Now and then a few test pieces were broken open to give an idea of what was taking place. At the end of the time mentioned a large number were broken open and examined, the amount of the corrosion determined and the general condition of the concrete noted. A most conspicuous feature of the results of this test, and a very surprising one in view of the results previously obtained at higher voltages, is the fact that cracking almost universally failed to occur. Of the 90 test pieces only 3 had cracked at the end of 5,500 hours. One of these was made of Brand B White Portland cement, a cement which was shown later to give much larger amounts of corrosion under certain circumstances than most of the ordinary brands of Portland cement under similar conditions, and the other two of Brand A

cement to one of which a foreign ingredient (20 per cent paraffin in gasoline) had been added. Both of the test pieces of Brand A cement which cracked possessed rather unusual characteristics and behaved very differently from their fellows of the same composition and must be regarded as abnormal test pieces.

It seems allowable, therefore, to say that not one of 87 normal test pieces made of ordinary Portland cement and put under test on 15 volts or less had cracked at the end of 5,500 hours. Of the test pieces which were then opened for examination it was found that in every case the concrete was broken with difficulty and appeared to be as sound as that of similar test pieces made at the same time and not subjected to the action of electric currents. In practically all cases there was more or less corrosion around the anode at the point where it entered the concrete and in some instances for an inch or so below the surface. In some cases, also, where there were voids in the concrete next to the iron there was some rust and slight pitting, but in nearly all cases where the concrete was in actual contact with the iron the corrosion was quite small and in many cases the iron was practically as bright and clean as when placed in the concrete. Condensed data of the tests on a few representative test pieces broken open to date are given in Table II. It is important to note that the total number of ampere hours per square inch of embedded electrode surface carried, on the average, is considerably larger than the corresponding figures for the high voltage test pieces at the time they cracked, the average being 2.6 for the low voltage, as against 0.83 required to crack for high voltage. It is evident, therefore, that the quantity of electricity that passes through a test piece does not alone determine the amount of damage that it may do, but that the rate at which the current flows is also an important factor. Moreover, it must be evident from these observations that the rate at which damage occurs decreases with decreasing voltage much more rapidly than the voltage is lowered, since in the present instance a reduction of the voltage to one-fourth of the value used in the high voltage tests enabled the test pieces to run with little or no damage for a period over 200 times as long as was required to destroy completely the specimens on the higher voltage.

TABLE II.—LOW VOLTAGE TESTS ON REINFORCED CONCRETE WITH IRON ELECTRODES.

Test Piece No.	Voltage.	Total Ampere Hours.	Ampere Hour Density, per sq. in.	Resistance in ohms.		Hours of Test.	Brand of Cement used.	Electrode used.	Age of Test Piece, days.	Corrosion.
				at Beginning of Test.	at End of Test.					
12	15	49.5	1.81	75	10,000	5,508	D	1-in. pipe	30	none
13	15	44.0	1.61	85	9,370	5,508	"	"	"	"
14	15	105.6	4.20	64	3,300	5,500	E	"	"	slight
15	15	53.4	1.95	75	21,000	5,500	"	"	"	"
16	15	45.4	1.66	94	3,000	5,500	C	"	34	"
17	15	39.9	1.28	81	50,000	5,500	"	"	"	"
18	15	55.0	2.10	75	12,500	5,500	F	"	24	"
19	15	62.3	2.30	69	15,000	5,500	"	"	"	"
20	15	74.3	2.73	100	5,000	5,500	G	"	23	severe
21	15	91.0	3.30	92	7,500	5,500	"	"	"	"
22	15	35.5	2.10	141	8,800	5,300	H	¾-in. round iron	46	none
23	15	35.2	2.10	133	15,000	5,300	"	"	"	"
24(*a*)	15	93.0	5.60	127	1,875	4,400	B	"	57	very severe
25	15	66.0	4.00	134	5,000	4,400	"	"	"	"
26	15	26.6	0.97	107	7,500	1,800	C	1-in. pipe	270	slight
27	15	47.6	1.74	88	7,500	1,800	"	"	"	"
28(*b*)	15	52.4	1.90	75	5,000	5,691	A	"	57	medium
29(*b*)	15	54.6	2.00	75	5,000	5,691	"	"	"	"
30(*c*)	15	121.9	4.50	81	469	2,477	"	"	"	very severe
31	5	11.7	0.43	126	5,200	5,300	"	"	80	none
32	5	13.7	0.50	137	5,700	5,300	"	"	80	"

(*a*) Hours to cracking, 732; Resistance at cracking, 470 ohms; Ampere hours per sq. in. to cracking, 2.4.

(*b*) Electrolyte distilled water.

(*c*) Hours cracking, 1000; Resistance at cracking, 260 ohms; Electrolyte, distilled water.

5. TESTS ON MATERIALS OTHER THAN IRON AS ANODE.

(*a*) *Copper, brass, carbon and copper-clad steel.*

A number of test pieces were made up with Brand A cement in a 1 : $2\frac{1}{2}$: 4 mixture, but using copper, brass, carbon and copper-clad steel as electrodes. Part of these were placed in circuit with the metal or carbon as anode, and subjected to voltages ranging from 15 to 60, for periods varying from a few days to 10 months. The data obtained from these tests are summarized in Table III. There was in all cases considerable corrosion of the embedded metal, but in no instance was the concrete cracked or otherwise injured. The fact that the concrete did not crack in any of the above cases is significant, and it is interesting to note in this connection that in the case of the metals used there is a tendency for the formation of soluble salts as the end products of corrosion. Since these salts remain in solution to some extent they diffuse through the concrete and thus do not give rise to the local mechanical pressure found in the case of iron where the insoluble oxides are precipitated near the surface of the anode. This phenomenon supports the theory that the cracking of the concrete is due to a mechanical pressure developed by the formation of oxides at the anode surface. This is referred to again in a later section dealing in detail with the causes of fracture in the concrete.

(*b*) *Aluminum.*

The possibility that the protective film formed over the surface of an aluminum covered rod, when embedded in concrete and made anode, might serve as a preventive of electrolysis in concrete was first suggested by Magnusson and Smith, and the fact that the experimental results published by them tended to show that aluminum would not suffer deterioration as iron does when maintained anode in concrete led to the following investigations.

The tests were made on aluminum rods with voltages ranging from 5 to 115, the embedded metal being made anode in some cases and cathode in others. In all cases there was rapid deterioration of the aluminum. In the cases in which the aluminum was positive there was corrosion of the metal, accom-

TABLE III.—ANODE EFFECTS IN CONCRETE USING BRASS, COPPER AND CARBON ELECTRODES.

Test Piece No.	Voltage		Total Ampere Hours.	Ampere Hour Density, per sq. in.	Resistance in ohms.		Hours of Test.	Electrode used.	Age of Test Piece.	Anode Loss, grams.
	at Beginning of Test.	at End of Test.			at Beginning of Test.	at End of Test.				
33	52.5	8.0	108.0	10.1	55	920	3,988	½-in. copper	24 days	8.6
34	60.5	104.0	108.0	10.1	63	11,900	3,988	½-in. brass	"	11.1
35	57.0	57.0	58.8	5.3	112	15,000	1,836	"	7 mos.	7.5
36	57.0	57.0	53.6	4.8	112	15,000	1,836	"	"	5.1
37	57.0	57.0	103.0	9.4	112	8,142	1,836	½-in. copper	"	9.8
38	57.0	57.0	95.6	8.3	112	8,142	1,836	"	"	10.0
39	15.0	15.0	31.6	3.95	250	9,370	5,643	⅜-in. brass	50 days	1.0
40	15.0	15.0	42.6	5.3	103	3,750	5,643	⅜-in. copper	"	....
41	15.0	15.0	33.3	2.0	115	3,000	1,808	½-in. brass	9 mos.	2.0
42	15.0	15.0	53.4	3.3	115	3,000	1,808	½-in. copper	"	3.5
43	57.0	57.0	75.4	9.1	120	2,110	1,302	⅜-in. copper clad steel	26 days	10.9
44	57.0	57.0	75.4	9.0	116	2,110	1,302	"	"	11.6
45	58.0	57.0	80.5	7.3	96	613	450	½-in. carbon	10 mos.	....
46	58.0	57.0	58.0	5.2	83	613	450	"	"	....
47	57.0	57.0	73.5	6.7	44	295	714	"	5 days	....
48	57.0	57.0	64.9	5.9	40	375	714	"	"	....
49	15.0	15.0	32.3	3.0	75	1,875	3,229	"	50 days	....
50	15.0	15.0	52.6	4.8	79	3,750	1,919	"	10 mos.	....
51	15.0	15.0	32.3	2.9	101	3,750	1,919	"	"	....

panied by a swelling action which in three of the five test pieces was sufficient to crack the blocks.

When the aluminum electrode was made negative on 15 volts the test pieces cracked at the end of 144 hours and when opened, the metal was found to be badly pitted, but the corrosion was different from that found in the positive test pieces. The decomposition products were black, forming a hard shell around the electrode from which they were separated very readily. This corrosion was doubtless due to a secondary action and in the light of experiments described later it seems quite probable that the action is the result of a concentration of alkali metals (sodium and potassium) near the negative electrode with the resulting formation of sodium and potassium aluminates. The dark color is doubtless due to the presence of iron as impurity. A check test piece made at the same time as the others and kept in water without current flow showed no corrosion whatever with the exception of a slight reaction between the metal and the alkali in the cement forming a very thin, hard layer on the surface.

6. CATHODE EFFECTS.

In the preceding section the phenomena noted are those resulting when current flows from the embedded iron or other metal out into the concrete. When the direction of current flow is reversed, making the iron cathode, very different effects are produced. In this case there is no tendency for the iron to corrode because of the current flow, but on the contrary the iron is protected from any natural corrosion that might tend to take place. In the published work of previous investigators no mention is made of any injurious effects either to the iron or the concrete in those test pieces in which the current flowed from concrete to iron, except in a single instance in which the test piece cracked. The cracking in this isolated instance was beyond doubt merely incidental, since all other investigators have failed to note any tendency to crack when the iron is made cathode. The conclusion has therefore been widely accepted that when the current flows from concrete to iron no effects are produced, and at the time these experiments were begun there appeared to be no substantial ground on which to question this conclusion. It was deemed advisable, however, to confirm these observations,

and accordingly a number of test pieces were made exactly similar to those used in the anode tests above described, using not only iron, but also brass, copper and carbon as electrodes. These were placed in circuit with the current flowing from the concrete to the electrode. Both high and low voltages were used and the conditions in general were kept exactly the same as with the anode tests except for the direction of current flow. Condensed data on these tests are given in Table IV. It will be noted that there is a rise in resistance as the test proceeds, very similar to that which occurs in the anode test pieces, but this rise is less marked, the average ratio of increase being about 10 to 1. The first 4 test pieces represented in the table (58–61) contained iron electrodes. Each one was started in series with an anode test pieces and the voltage went down to about nine volts at the end of 1,200 hours due to the relatively greater rise of resistance of the anode test pieces. The 4 were then connected in series on 115 volts and the voltage for the remaining 7,700 hours of the test remained practically constant at the values given in the table for the voltage at the end of the test. Throughout the test water was forced out around the embedded iron and kept the top of the test piece wet. This forcing out of water was evidently due to the formation of gas at the cathode, which, on escaping, forced the water through the pores of the concrete to the surface. This water carried calcium hydroxide in solution and as evaporation took place calcium carbonate was deposited in rings surrounding the cathode on the surface of the concrete. The greater part of the gas evolved was hydrogen. As the test proceeded the mortar on the top surface of the concrete immediately surrounding the cathode became quite soft. This softening extended in the surface to a distance of about $\frac{3}{4}$ in. from the cathode and in this region the concrete was darker than elsewhere. After the expiration of several months, and at intervals thereafter, certain of the test pieces were broken open for examination. In every case the concrete blocks were broken with diffculty, the main body of the concrete being apparently as sound as in similar test pieces not subjected to the action of electric currents. On laying the test piece open it was found that the embedded metal was in a perfect state of preservation, but the entire region surrounding the cathode for a distance of $\frac{1}{6}$ to $\frac{1}{4}$ in. from the surface of the

metal was considerably darker in appearance than the main body of the concrete and was very soft, like the concrete immediately surrounding the cathode at the surface. The cement here could be shaved off with a knife like soft soapstone.

In test pieces 62 and 63, having copper and brass cathodes respectively, the same phenomena were observed, except that in test piece 63 no water was forced out at the surface surrounding the cathode. Both cathodes were quite slippery when first removed from the concrete. Test pieces 64, 65, 66 and 67 contained iron cathodes as above, but were subjected to only 15 volts throughout the test. The results were the same as described above, except that they had not advanced quite as far. Test pieces 68 and 69, containing copper and brass electrodes respectively, behaved exactly the same as 62 and 63, except that the resulting effects were less pronounced. Test piece 70 contained an electrode consisting of a piece of arc light carbon. The carbon itself disintegrated badly and after the bond had been destroyed it was forced out of the concrete by the formation of gas underneath. The mortar near the carbon was found to be disintegrated in the same manner and to about the same extent as in the case of metal cathodes on 15 volts.

In all of the above test pieces there was a rather sharp line of demarcation between the softened area and remainder of the concrete, the soft portion being in every case readily distinguishable by its darker color. Outside this darkened zone the concrete appeared to be as sound as in test pieces not carrying current, and no physical change of any kind could be detected therein. After the test pieces had been broken open for some time and allowed to dry, the darkened zone became somewhat lighter in shade, but there always remained a distinct difference readily detectable by the eye. The softening also diminished greatly as the concrete became drier, and after becoming thoroughly dry it became nearly as hard as the unaffected mortar but remained distinctly more friable.

7. BOND TESTS OF CATHODE TEST PIECES.

The disintegration of the mortar in the immediate vicinity of the cathode, as described above, led to tests on the relative shearing strengths of the bond where current had passed with

TABLE IV.—CATHODE EFFECTS.

Test Piece No.	Voltage		Resistance		Total Ampere Hours.	Ampere Hour Density, per sq. in.	Hours of Test.	Electrode used.	Age of Test Piece, days.	Brand of Cement used.
	at Beginning of Test.	at End of Test.	at Beginning of Test.	at End of Test.						
56	15.0	15.0	130.0	150	.....	....	144	½-in. aluminum	36	A
58(a)	57.5	20.0	67.0	800	407.0	25.4	8,900	¾-in. round iron	40	"
59	58.5	30.5	73.0	950	395.0	24.7	8,900	"	"	"
60	59.0	17.0	61.0	1,650	415.0	25.7	8,900	"	"	"
61	52.0	45.0	60.0	2,100	422.0	26.2	8,900	"	"	"
62	56.0	96.0	56.0	1,170	387.0	36.8	3,988	½-in. copper	21	"
63	57.0	16.0	57.0	195	387.0	36.8	3,988	½-in. brass	"	"
64	15.0	15.0	53.6	357	330.6	12.2	5,508	1-in. pipe	34	D
65	15.0	15.0	53.6	384	345.0	12.8	5,508	"	"	E
66	15.0	15.0	57.6	405	215.0	8.0	5,508	"	33	C
67	15.0	15.0	57.0	300	387.8	14.3	5,808	"	"	F
68	15.0	15.0	82.0	468	263.5	35.1	6,200	⅜-in. copper	50	A
69	15.0	15.0	88.0	500	279.4	36.0	6,200	⅜-in. brass	"	"
70	15.0	15.0	75.0	208	219.0	20.7	3,229	½-in. carbon	"	"

(a) Resistance after 20 minutes, 300 ohms. Loss in weight of electrode, 5 grams. Hours to cracking, 216.

embedded iron cathode and where it had not. For this test pieces 58, 59, 60, 61 of Table IV were used together with 4 identical test pieces through which no current had passed. In order to carry out this mechanical test the bottoms of the saturated test pieces were ground over a surface perpendicular to the axis of the embedded iron rods until the lower ends of the rods were exposed. The projecting ends of the rods were then sawed down to a length of about 2 in. and the test pieces placed, one at a time, in a testing machine which pushed the rods through the blocks sufficiently to give the maximum shearing strength of the

TABLE V.—BOND STRENGTH OF CATHODE TEST PIECES ON HIGH VOLTAGE.
(Numbers correspond with numbers in electrical test record sheet.)

Test Piece, No.	Total Load, lbs.	Friction Load, lbs.	Breaking Strength of Bond, lbs. per sq. in.	Friction Strength of Bond, lbs. per sq. in.	Area of Embedded Iron, sq. in.
58	2,170	1,200	127	70	17.0
59	2,640	1,500	157	90	16.8
60	2,550	1,270	155	77	16.4
61	1,300	1,000	79	61	....
Average .	.....	.....	129	74	
Bond Strength of Test Pieces through which no Current had Passed.					
58	14,500	7,000	800	400	17.5
59	9,600	3,800	585	231	16.4
60	10,850	8,800	711	516	15.1
61	8,340	6,000	496	357	16.8
Average .	.....	.....	648	376	

bond and also the friction load after the bond was broken. The test was carried out in the usual way, care being taken to see that the rod was upright on the platform of the machine, and the load gradually run up until the beam dropped and slipping commenced. The load at which the beam dropped was taken as the breaking load and a succeeding reading made while the rod was slipping through the block, was taken as the friction load. The rod was pushed through the block a distance of about 0.1 in.

The results are given in Table V. It is there seen that the passage of current under the conditions of the test carried out on test pieces 58, 59, 60 and 61, reduced the bond to about $\frac{1}{5}$ of its original value.

The electrical test here was quite severe, but the other test pieces which were run at voltages ranging from 5 to 15 as described, showed on examination that the same disintegrating action was going on and the extent of the softened area, while somewhat irregular and slightly indefinite, indicated that at least during the first few months of a test the amount of this disintegration is probably roughly proportional to the quantity of electricity that has passed through the concrete and is thus but indirectly affected by the voltage used. In this respect it is in marked contrast to the anode effects previously described, which appear to diminish much more rapidly than the voltage, until at voltages of 5 to 10 on the test pieces used they practically disappear.

8. CAUSE OF DISINTEGRATION OF THE CONCRETE ABOUT THE CATHODE.

A number of experiments were carried out to determine the cause of the softening of the concrete near the negative terminal. The details will not be given here, but it was found that all of the test pieces contained considerable amounts of sodium and potassium in soluble form. When current passed through the test piece these alkalies were gradually concentrated near the negative terminal and in time the concentration of the alkali became sufficient to attack the cement, the principal reactions being the replacement of the calcium in the calcium silicates and aluminates by sodium or potassium with the formation of soluble sodium or potassium silicates and aluminates and the formation of calcium hydroxide. Figs. 4 and 5 show the general appearance of the test pieces after being broken open. As indicated by the cards accompanying each specimen some of the specimens were made up with NaOH and KOH added to the water used in making the mortar. The percentages are expressed in terms of the amount of cement used.

9. EFFECT OF ELECTRIC CURRENT ON THE MECHANICAL STRENGTH OF PLAIN CONCRETE.

While the foregoing experiments appear to show quite clearly that no effect is produced by the passage of an electric current through the main body of the concrete remote from the elec-

trodes, it was deemed advisable to corroborate this with a series of mechanical tests on test pieces so designed as to eliminate the electrode effects. The only mechanical test of this sort that has been published was conducted by Magnussen and Smith. Although their test was continued but a short time it appeared to show that under the conditions of the test there was no appreciable decrease in the crushing strength of the mortar.

In order to secure further information on this subject a test was carried out using concrete cylinders 6 in. in diameter by 8 in.

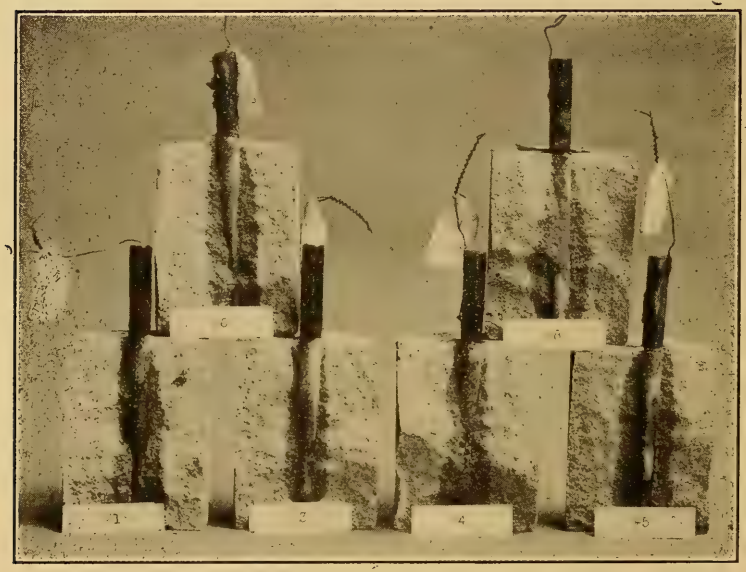


FIG. 4.—EXTENT OF DISINTEGRATION ABOUT THE CATHODE.

long with no iron embedded. These cylinders were made of $1 : 2\frac{1}{2} : 4$ concrete, using Brand A cement for part of them and Brand C for the rest; sand and crushed trap rock constituted the aggregate. After the concrete had become thoroughly set 18 of the cylinders were provided with electrical connections by plastering 3 x 3-in. carbon plates to the ends of each cylinder with cement mortar. A copper lead was attached to each carbon plate. When the mortar which held on the electrodes had set, the cylinders were laid on 2 x 2-in. strips of wood and the copper

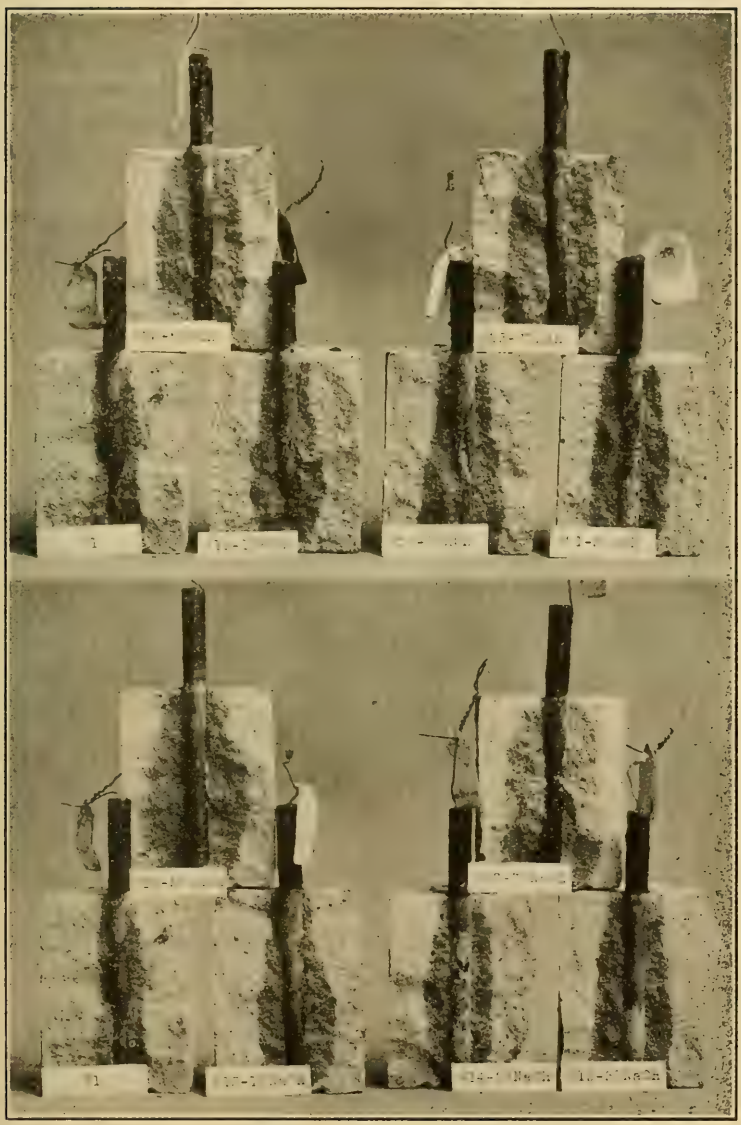


FIG. 5.—EXTENT OF DISINTEGRATION ABOUT THE CATHODE.

leads connected to the terminals of a 115-volt circuit. The potential gradient impressed on these specimens was therefore 172 volts per foot and the test was extremely severe. Check test pieces were placed in the immediate vicinity and all of them were thoroughly wet down each day with tapwater, so that both the check test pieces and those carrying current were under the

TABLE VI.—EFFECT OF ELECTRIC CURRENT ON MECHANICAL STRENGTH OF CONCRETE.

Carried Current.		No Current.	
Made with Brand A Cement.			
Test Pieces No.	Crushing Load.	Test Pieces No.	Crushing Load.
173	60 630	185	73 270
174	69 210	186	75.840
175	66,910	187	51,000
176	64,060	188	80,890
177	69,700	189	56,210
178	53,650	190	57,050
179	41,100	191	63,020
180	66.300	192	53,490
181	69,850	193	65,250
182	73,450	194	55,310
183	92,900	195	86,110
184	31,350	196	67,100
		197	51,210
		198	88,630
Average	63,260	Average	66,002
Made with Brand C Cement.			
199	47,400	205	42,000
200	46,000	206	35,440
201	31,450	207	38,550
202	50,750	208	45,040
203	45,660		
204	49,300		
Average	43,400	Average	40,250

same moisture conditions. Current flowed continuously for 14 months and averaged 0.009 ampere for each test piece. This means that approximately 75 ampere hours of electricity passed through each under a potential gradient of a little more than 14 volts per inch. The current density averaged about 0.0003 ampere per sq. in. of cross-sectional area of the test piece.

At the end of the period of 14 months the test pieces were removed from the circuit and tested for their crushing strength.

Those which had been subjected to current were prepared for the test by simply splitting off the plaster which adhered to the ends. No grinding was required. The testing was done with a 200,000-lb. testing machine. Blotting paper of 7 or 8 thicknesses was placed on the bottom and top of each test piece in order to insure uniform distribution of pressure and the load was run up to a maximum shown by the dropping of the beam when the test piece failed. Nearly all of the failures were of such a character as to show the usual 45 deg. shear. The results are given in Table VI. There is no indication there that the passage of current seriously affected the crushing strength of the concrete. It will appear from the first series of tests, made with Brand A cement, that the averages show a slight difference in favor of the test pieces which carried no current. In the second series, however, using Brand C cement, the reverse is the case, the difference being in favor of the test pieces carrying current. The differences are negligibly small, however, in view of the wide variation between individual test pieces subjected to the same treatment. These results have since been confirmed by additional experiments which are not described in detail here.

10. CAUSE OF CRACKING OF REINFORCED CONCRETE BY ELECTRIC CURRENTS.

The merits of the different theories advanced in explanation of the cause of the cracking of concrete around the positive electrode will not be discussed here. It is evident, however, that the theory which attributes the cracking to the formation of iron oxides between the iron and the concrete is the correct one. Of the numerous experiments carried out to show this but one will be described here.

A cast iron shell, $1\frac{1}{2}$ in. in external diameter, was made up as shown in Fig. 6. This cylinder was slotted lengthwise at opposite extremities of a diameter to a depth $\frac{1}{16}$ in. less than the thickness of the shell. The thickness of the shell was $\frac{1}{4}$ in. in this case, so the cuts were each $\frac{3}{16}$ in. deep, leaving a thickness of $\frac{1}{16}$ in. of cast iron holding the two halves of the shell together on either side. The object in making slots of this character in the shell was to cause practically all of the elongation of the iron due to an internal pressure to take place in the slot. The smaller cross-sectional area of iron there would then reach its elastic limit

sooner than the thicker portion and lead to a quick break in the event of the pressure becoming great, while a shell $\frac{1}{16}$ in. in thickness throughout might expand to such an extent that a break would not occur. A $\frac{3}{4}$-in. solid iron electrode was cemented in the bore of the shell with 1 : 1 mortar made up with a 3 per cent NaCl solution. After the mortar had set the shell was put in circuit on 15 volts with the central electrode positive. In about 3 days the two halves of the shell were found to be broken apart by the formation of rust on the surface of the positive electrode.

The diameter of the rod was $\frac{3}{4}$ in. and the length $1\frac{1}{2}$ in. The area of the broken section of cast iron was found to be 0.106 sq. in. and its modulus of rupture about 20,000 lb. per sq. in. From these data the actual pressure which had developed on the surface

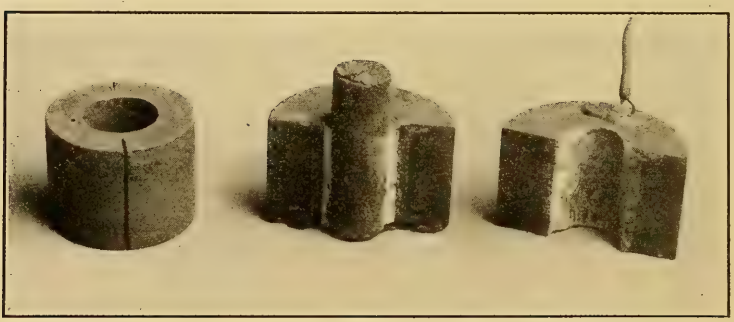


FIG. 6.—TEST PIECE FOR DETERMINING CAUSE OF CRACKING CONCRETE BY ELECTRIC CURRENTS.

between the mortar and the rod at the time of rupture can be calculated and the result in this case is 3,760 lb. per sq. in. This pressure is much more than ample to produce the cracking of concrete that has been observed in past experiments, and appears therefore to be a sufficient explanation of the phenomenon.

11. THE RISE OF RESISTANCE OF REINFORCED CONCRETE DUE TO FLOW OF CURRENT.

(*a*) *Rise of Resistance in Anode Test Pieces.*

The electrical resistance of concrete is a factor of great importance but one which varies so greatly with varying physical conditions, especially with varying moisture content, that it is difficult to give any reliable numerical values except in the case

of saturated specimens. An average value for representative specimens of thoroughly wet concrete may be taken as varying from 4 to 6,000 ohms per cu. cm., depending somewhat on the proportions. With reduced moisture content the resistance rises rapidly, the concrete becoming a fairly good insulator when thoroughly dry.

In the published work of previous investigators mention is frequently made of the fact that when electric current passes through reinforced concrete, under the conditions usually imposed upon it in the laboratory, there is a gradual increase of resistance with time, regardless of the direction of flow of current. Nothing definite has been said as to the probable cause of this rise of resistance, nor does it appear that its importance as a factor in minimizing trouble from electrolysis in reinforced concrete has been fully appreciated. With a few exceptions, which are considered later, such a rise of resistance occurred in all of the test pieces used in the course of this work. When they were first placed in circuit the resistance was a minimum, but upon the application of the electric current the resistance began to rise. In the low voltage test pieces the first apparent increase in resistance may have been more or less affected by polarization, but in the higher voltage tests at 15 to 100 volts, polarization effects are negligible. The resistance at first increased quite rapidly, but the rate of increase usually commenced to diminish after about 10 to 12 hours. The rise of resistance continued for a period varying from several days to several months according to circumstances, until an approximately constant condition was reached. In the case of higher voltages the increase of resistance was the more rapid at first, but the rate of increase was not maintained in proportion to the voltage applied so that a greater ampere hour flow of current was required to produce a given rise of resistance in the case of high voltage than where the voltage was low. Typical curves showing the resistance as a function of time and also as a function of the ampere hour flow of current are shown in Figs. 7, 8 and 9. The relative magnitude of the initial and final resistances varied very much under different conditions but the general tendency was the same whether the test pieces were run as anode or cathode on high or low voltage. The results of the tests for anode effects on low voltage, which are summed up in Table II, show an average increase of resistance

to 137 times the original value at the end of 5,500 hours. The results of the tests for cathode effects on all voltages, which are summed up in Table IV, show an average increase to 14 times the original resistance at the end of about the same period. The anode tests on high voltage do not show such a large increase of resistance as the anode tests on low voltage; but this is due, no doubt, to the fact that owing to the early destruction of the test pieces on high voltage, these tests were of comparatively short duration.

A rise of resistance of sufficient magnitude to cause the reduction of current through a specimen of reinforced concrete to less than 1 per cent of its original value was recognized as a phenomenon of great importance and was therefore given special consideration.

(*b*) *Cause and Location of the Rise of Resistance in Reinforced Concrete due to Current.*

All the details of the experiments along this line cannot here be discussed, and the results are merely summarized. By measuring potential drops between various points between the anode and cathode, care being taken to eliminate effects of polarization at the electrodes, it was found that most of the rise of resistance was at the surface of the concrete nearest the cathode and is due to a film which forms on, and perhaps to some extent within, the surface of the concrete. Analysis shows that there is a large accumulation of calcium carbonate at this point and the cause of the rise of resistance is evidently due to the concentration of $Ca(OH)_2$ near the cathode surface by the current, where it comes in contact with CO_2 absorbed by the water from the air with the resulting precipitation of $CaCO_3$ within the pores of the concrete near the surface of the test piece, thus plugging up these pores and forming a nearly impermeable wall.

(*c*) *Rise of Resistance in Cathode Test Pieces.*

In case the embedded electrode is made cathode the rise of resistance, while still greatly in evidence, is much less marked that when the embedded electrode is anode, as noted below. This might be expected from the fact that CO_2 has very little access to the $Ca(OH)_2$ at the surface of the embedded electrode and hence the tendency to produce a dense precipitate of $CaCO_3$

would be very slow. In fact, there is good reason to believe that the rise of resistance which takes place at the embedded cathode is due chiefly to the liberation of gases at that point by the electrolysis of water, and since these gases are necessarily slow in escaping through the pores of the concrete, the electrolyte is largely excluded from the pores near the cathode surface with

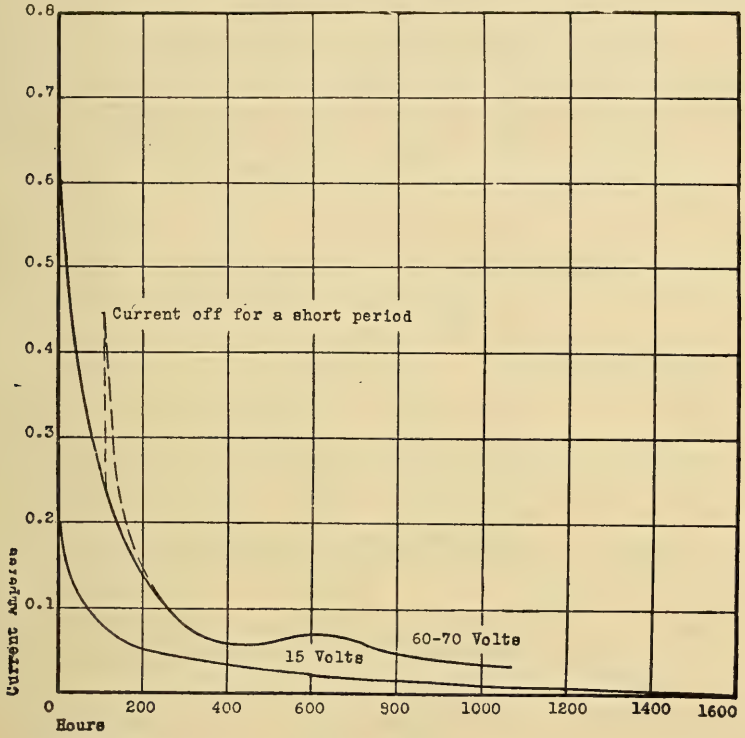


FIG. 7.—CHARACTERISTIC CURRENT-TIME CURVES OF HIGH AND LOW VOLTAGE TEST PIECES WITH IRON ANODE.

consequent rise in resistance. This is indicated by bubbling and forcing out of water around the cathode, by variations in the resistance of such test pieces of as much as 10 to 20 per cent while current is flowing constantly, and also by the fact that the resistance drops back quite close to its original value when the current is discontinued for a few days and the gas permitted to diffuse.

(d) Current-Time, Resistance-Time, and Resistance-Ampere Hour Characteristics of Concrete Test Pieces.

Some very interesting curves are shown in Figs. 7, 8 and 9. One of the curves of Fig. 7 is a plot of the current values for a high voltage anode test piece as a function of time and the other the same for a low voltage anode test piece. These curves cover

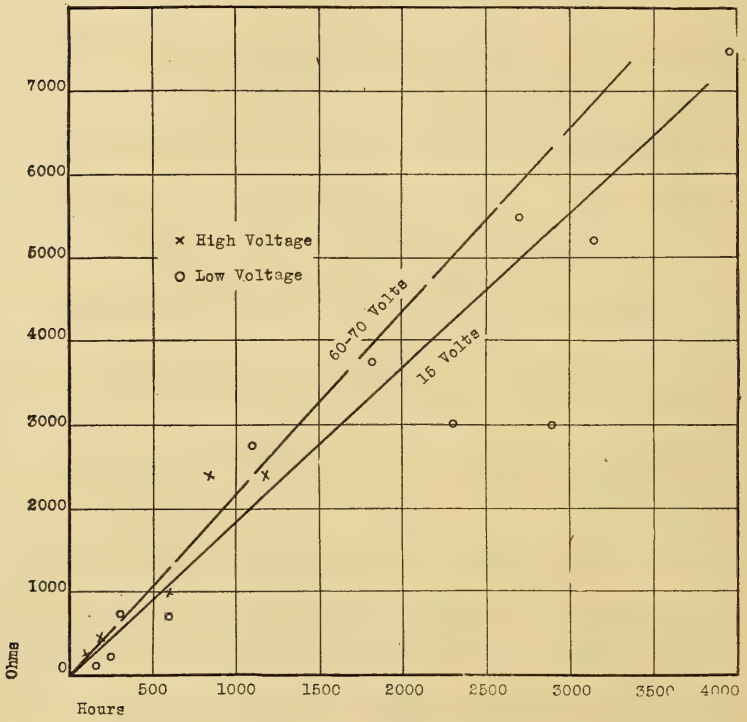


FIG. 8.—RESISTANCE OF HIGH AND LOW VOLTAGE TEST PIECES WITH IRON ANODE, AS A FUNCTION OF TIME.

the first 1,000 hours of each test. It is seen that the greatest drop of current occurs during the first 200 hours. It is probable that this is due to gas forming quite rapidly with the comparatively high current density at the beginning of the test and clinging in a film to the outer electrode, because if the current is then discontinued for a time long enough to allow the gas to

diffuse thoroughly the resistance decreases almost to its original value; which is partially shown in the curve of the high voltage test piece at the point where the current is indicated as having been off for a short time—an hour or two at most. Beyond the 200-hour point the curves are both comparable with a straight line and nearly parallel. This is also shown to some degree in the curves of Fig. 8. These show plots of resistance as a function of time for a high and a low voltage test piece respectively. The high voltage test piece was continued in circuit but a short time in comparison with the one at low voltage but there is an indication that the curves would be very nearly parallel if they had

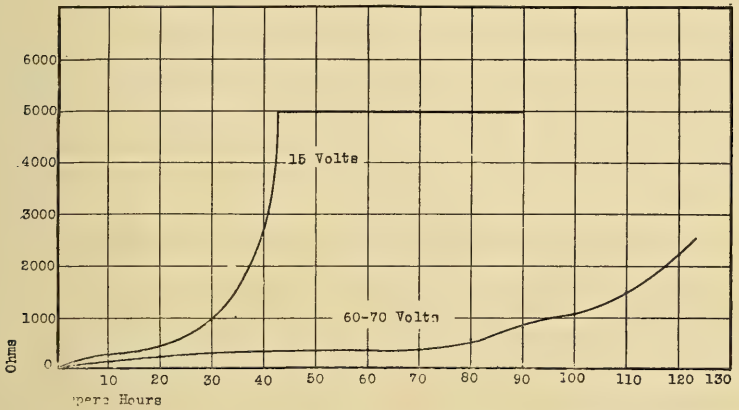


FIG. 9.—RESISTANCE OF HIGH AND LOW VOLTAGE TEST PIECES WITH IRON ANODE, AS A FUNCTION OF THE AMPERE-HOURS PASSED.

both been continued in circuit indefinitely. Fig. 9 shows plots of increase of resistance as a function of ampere hours for high and low voltage. If the continued increase of resistance after the formation of the gas film were wholly an electrical effect and its rate of increase depended on the rate of ampere-hour discharge, the two last-named curves should be expected to practically coincide. There is a wide divergence, however, showing that the increase of resistance is not wholly an electrical effect. A much larger amount of electricity passes on the high voltage for a given rise of resistance than on the low voltage. It would thus appear that lapse of time is an important factor in the rise of resistance

noted. Resistance measurements made on test pieces which do not carry current show, however, that the mere passage of time without current flow, has but little effect; a fact which is proved more fully by incidental measurements made on a number of other test pieces in the course of the work. The foregoing phenomena are entirely in accord with the explanation of the rise of resistance set forth above.

12. EFFECT OF ADDING SALTS TO CONCRETE ON RISE OF RESISTANCE.

A notable exception to the practically general rule with regard to the increase of resistance of reinforced concrete when current flows through it will now be considered. A not uncommon practice in connection with the placing of concrete in cold weather is to dissolve 2 or 3 per cent of salt (NaCl), or calcium chloride ($CaCl_2$), in the water which is used in making the mortar. It is not definitely known that this has any detrimental effect upon the concrete itself, but it is of material benefit in lowering the freezing point of the mixture while setting. The effect of this addition of salt to concrete in making, upon its electrical properties, seemed to invite investigation, so several tests were made in the usual manner with salt added in various ways. In all cases where such test pieces were used as anodes the damage was not only found to be very much hastened but instead of the usual increase of resistance with passage of current, the resistance actually decreased in some cases as the test proceeded. Two test pieces in particular which were made up with no addition of salt, but which were tested in a 3 per cent salt solution, decreased from 115 ohms to 80 ohms and 130 ohms to 60 ohms at the end of 410 hours and 1,490 hours, respectively.

A second test, in which the concrete was made up with a 10 per cent salt solution and tested as anode on 15 volts over a period of 5 days, showed a decrease of resistance of 25 per cent. Cathode test pieces treated in a manner similar to the tests on the first two test pieces mentioned showed an increase of resistance of 20 to 25 per cent in the same length of time. This marked difference in character from that of all the other test pieces in this work must be attributed chiefly to the salt, or, more broadly, to the presence of an excessive amount of the acid radical,

chlorine. The currents taken by the test pieces were rather high in most cases and hence heating may have had something to do with the decrease of resistance instead of an increase of resistance with time, but it cannot account for all of it. In tests where no salt was added the temperature rise was nearly as great but the rise of resistance never failed to occur.

The failure of the resistance of the concrete to rise when a large amount of sodium chloride is added may be explained in part by the fact that, owing to the relatively great concentration of the sodium ions as compared with the calcium ions in this case, the greater part of the current is carried by the sodium and hence little calcium is carried to the surface. The chief factor, however, is that he presence of sodium chloride in a solution prevents the precipitation of calcium carbonate at ordinary temperatures and thus the plugging up of the pores can not occur. This phenomenon is, therefore, a corroboration of the explanation above given for the rise of resistance of test pieces of normal concrete.

13. Rise of Resistance of Concrete with Damp Earth as Electrolyte.

It is evident that the rise of resistance of reinforced concrete takes place under all the conditions of the electrolysis experiments conducted in the laboratory with the exception of that of an appreciable amount of chlorine, or perhaps any other acid radical, being added to the cement. These laboratory conditions provide a liquid electrolyte in all cases. In practice wet soil would more often be the electrolyte if conditions were favorable to electrolysis. Current would also be likely to flow between sections of reinforcement without leaving the concrete. In order to determine whether or not this same rise of resistance would occur in the case of concrete embedded in soil, two blocks of concrete containing embedded iron were molded in holes dug in the earth so that about 3 ins. of the block projected above the surface. The blocks were of 1 : $2\frac{1}{2}$: 4 concrete, $3\frac{1}{2}$ ft. square by $2\frac{1}{2}$ ft. deep. Four electrodes were embedded perpendicularly to a depth of 2 ft. in each block with 4 or 5 in. of each one projecting for electrical contact purposes. These electrodes consisted of 1-in. iron rods, 1-in. iron pipe and a length of 4-in. wrought iron pipe.

The blocks were placed about 2 ft. apart. Three of the eight electrodes were connected to the negative terminal of the 15-volt circuit and the remaining 5 to the positive terminal. Current flowed continuously except for about 33 hours each week. Part of the current which flowed to the positive electrodes flowed out through the surface of the blocks to the soil and to a large ground plate nearby which was connected to the negative terminal of the 15-volt circuit. The rest returned by way of the electrodes which were connected to the negative terminal of the 15-volt circuit. Current readings were taken on each electrode from time to time. The values of the current on each electrode were plotted as a function of time and showed that the rise of resistance takes place but it is a longer time in appearing than in the case of cylinders tested in jars.

It is therefore seen that a great rise of resistance takes place under conditions practically identical with those occurring in practice and it is, therefore, an important factor in reducing damage to concrete structures and particularly so in the case of moderate or low voltages, such as would almost invariably be met with under actual conditions. Its magnitude is such as to increase by many times the life of a structure over what it would have if the rise of resistance did not occur.

14. VARIATIONS OF THE MAGNITUDE OF ELECTROLYTIC CORROSION OF IRON IN CONCRETE.

It is well known that in cases of corrosion by electric currents, the total amount of corrosion does not in general correspond with Faraday's Law, but is in many cases considerably less than the theoretical amount. In particular, in the case of concrete, it was to be expected that the actual amount of corrosion would be less than that expected from Faraday's Law, because of the alkalinity of the electrolyte. This obviously has an important bearing on the subject of electrolysis in reinforced concrete, because the extent of the reduction in the amount of corrosion determines the damage which will result to the reinforcing material by a given ampere discharge and also determines in greater or less degree the extent of the injury to the concrete itself.

The first data on electrolysis of iron in concrete that admitted of comparing the theoretical amount of corrosion which

should have occurred at the anode with that which actually did occur were obtained by Glauber and reported on by Prof. Langsdorf in 1909. The results were very indefinite but showed that the amount of corrosion actually obtained and that which should have been obtained according to Faraday's Law did not agree, the observed corrosion being only a fraction of the theoretical. In the terms of Faraday's Law the theoretical amount of metal which will dissolve from an iron anode in an electrolyte, a solution of ferrous sulphate for instance, is $\frac{2.090}{n}$ grams per ampere-hour where n is the change of valency. If the change of valency is 2, as it is in nearly all ordinary cases, the amount of iron dissolved per ampere-hour would be 1.045 g. However, if the proper conditions are not maintained in the cell while the process of electrolysis is going on, or if an alkaline solution is used, a quantity of metal considerably less than the theoretical may be dissolved; that portion of the electricity passed in excess of what is used in dissolving the iron being consumed in breaking up the electrolyte. The quantity of metal actually dissolved divided by the theoretical amount according to Faraday's Law is called the "efficiency of corrosion." The value of this term for electrolysis of iron in concrete, as stated above, was shown by the data in Langsdorf's report to be, in certain cases at least, but a few per cent. A similar result was reported by Burgess two years later. He also tested some specimens in a 3 per cent salt (NaCl) solution which gave the rather surprising result of increasing the efficiency of corrosion to percentages varying from 55 to 80. The exactness of the results in general was open to some question, however, because during the tests a considerable amount of natural corrosion could easily have taken place on the interior of the iron pipe which he used in his experiments and the method used for cleaning the pipes, which was by brushing, would necessarily give rise to some uncertainty. The experiments were of great value, however, for the purpose for which they were intended, that is, to point out the general effect.

By way of verification of the foregoing results, a number of tests were carried out using different cements and every effort was made to have the conditions of the tests as nearly uniform as possible. The test pieces used were made with considerable care and consisted of 2-in. cubes of 1 : 2 mortar made up wet with

standard Ottawa sand and distilled water. A round iron electrode $\frac{1}{4}$ in. in diameter by $2\frac{1}{4}$ in. long was embedded $1\frac{3}{4}$ in. deep in each cube, as shown in Fig. 10. Before embedding these electrodes they were filed to a bright surface and a short bare copper wire soldered to each one. This was followed by weighing them on a balance having a sensibility of 0.0005 g., the average weight of the anodes being about 14 g. In order to prevent natural corrosion of the exposed portion of the electrode while the test piece was immersed in distilled water undergoing the setting pro-

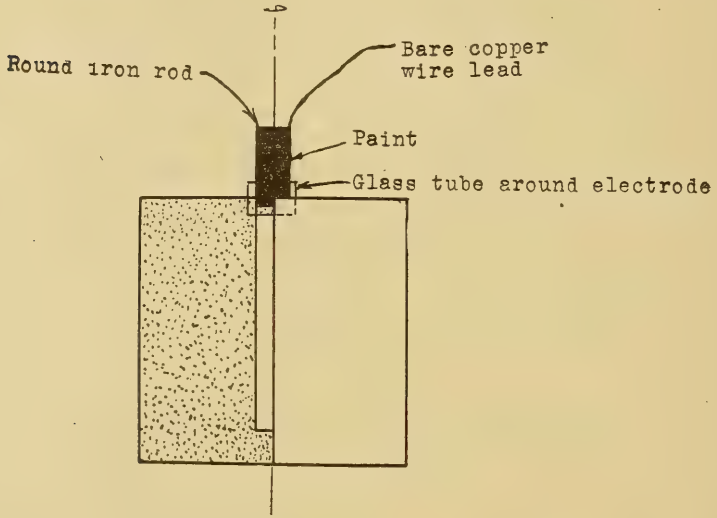


FIG. 10.—TYPE OF TEST PIECE USED FOR DETERMINING EFFICIENCY OF CORROSION.

cess, it was painted with two coats of an alkali resisting preservative paint.

After the setting process had continued for approximately two weeks the test was commenced. Each test piece was placed in a crystallizing dish 5 in. in diameter by 2 in. deep and surrounded by a sheet iron cylinder which served as the cathode. The dish was then filled to about $\frac{1}{4}$ in. from the top of the cube with distilled water, and the test piece put in circuit on 15 volts with the central electrode anode. The current density averaged

TABLE VII.—EFFICIENCY OF CORROSION IN VARIOUS CEMENTS.

Test Piece No.	Brand of Cement.	Voltage.	Ampere Hours.	Ampere Hours Density, per sq. in.	Anode Loss in m. g.	Theoretical Loss.	Efficiency of Corrosion, per cent.	Hours to Cracking.
77	C	15	3.95	3.10	0.085	4.11	2.07	202
78	"	"	2.72	2.10	0.032	2.84	1.12	202
79	"	0	0.00	0.00	0.001	0.00	....	Check
80	E	15	2.44	1.92	0.047	2.54	1.85	202
81	"	"	1.94	1.53	0.040	2.03	1.97	202
82	"	0	0.00	0.00	0.000	0.00	....	Check
9	A	15	7.70	1.30	0.046	8.00	0.50	No Cracking
10	"	"	"	"	0.068	"	0.85	...
11	"	"	"	"	0.118	"	1.47	...
12	"	"	"	"	0.223	"	2.80	...
83	"	"	0.99	0.78	0.054	1.03	5.20	42
84	"	"	1.16	0.91	0.079	1.22	6.40	66
85	"	0	0.00	0.00	0.003	0.00	....	Check
86	F	15	1.57	1.23	0.084	1.64	5.10	90
87	"	"	1.54	1.21	0.061	1.61	3.70	90
88	"	0	0.00	0.00	0.004	0.00	....	Check
89	I(a)	15	0.43	0.34	0.073	0.455	16.00	24
90	"	"	0.42	0.33	0.058	0.437	13.20	24
91	"	0	0.00	0.00	0.004	0.00	....	Check
92	J	15	4.83	3.80	0.017	5.04	0.33	No Cracks
93	"	"	5.74	4.52	0.023	6.00	0.38	"
94	"	0	0.00	0.00	0.000	0.00	....	Check
95	K(b)	15	1.52	1.20	0.237	1.58	15.00	193
96	"	"	1.58	1.24	0.160	1.65	9.70	193
97	"	0	0.00	0.00	0.001	0.00	....	Check

(a) Cement contained 0.3 per cent SO_3.
(b) Erz cement.

about 0.025 amperes per sq. in. of anode surface at the beginning and gradually decreased to about 0.005 ampere per sq. in. at the end of the test. Current flowed continuously with the exception of 33 hours each week, current readings being taken at intervals which increased as the test proceeded. When cracking occurred, or, if cracking did not occur, as soon as a sufficient number of ampere hours had passed to give a reliable indication of the efficiency of corrosion, the test pieces were removed from the circuit and broken open. The corroded electrodes were cleaned by an electrolytic process, then weighed and the losses determined. Table VII contains the condensed data. The test of each cement was made on a set of 3 cubes, one being kept without current as a check on natural corrosion and the other two tested electrically as described.

The check test pieces showed very little natural corrosion, the losses in these cases being hardly greater than the errors which would be expected to occur in weighing. The efficiencies of corrosion compare very well with those obtained by Burgess and show beyond a doubt that under the conditions of these experiments but a small fraction of the current passed through a test piece of reinforced concrete is effective in dissolving their on. The variations in efficiency of corrosion between different cements are quite marked; the corrosion efficiency of some test pieces of Brands A and F being 3 or 4 times that of Brands C and E. Brand J shows a remarkably low efficiency of corrosion but while under test the resistance of the test pieces was less than half that of the others. Brand I cement contained only 0.3 per cent SO_3, no calcium sulphate being added in grinding the particular batch from which the test pieces were made. Brand K cement shows a comparatively high efficiency of corrosion. This cement contains a relatively large percentage of iron; the iron replacing part of the aluminum which enters into the composition of ordinary Portland cement.

An examination of the chemical analyses of these cements shows that there is apparently no relation whatever existing between the variations in the efficiency of corrosion and the SO_3 content of a cement. This is not surprising, however, when the fact is taken into account that in every case the mortar contains more than enough $CaSO_4$ to saturate the absorbed water.

The variations in efficiencies of corrosion are, therefore, quite unaccountable on that basis.

One of the most important facts to be noted in connection with the efficiency of corrosion in concrete is the great change in the efficiency of corrosion attendant upon a change of the voltage through a wide range of values and the consequent accompanying change of current density at the surface of the electrode. There is good evidence of this change in the efficiency of corrosion in the results of the anode tests described in the first part of this paper. The high voltage tests recorded in Table I show that cracking of the test pieces occurred after an ampere hour density of 0.83 per sq. in. had passed through them. Test pieces 10 and 11 of Table I were in such a condition at the end of the test with regard to natural corrosion of the exposed iron as to permit cleaning and weighing to determine the loss by electrolysis. Test pieces 10 and 11 showed an efficiency of corrosion of 37 per cent and 40.8 per cent respectively. There is a marked contrast between these values and those found in Table II where the results of testing on 15 volts are recorded. The 15-volt test pieces passed an average of 2.6 ampere hours per sq. in. of anode surface with no sign of cracking, excepting three isolated instances out of ninety, the characteristics of which were such as to exclude them from consideration as normal. The electrode of test piece No. 25 of Table II was undoubtedly corroded more than that of any test piece recorded there. It was cleaned and weighed and the efficiency of corrosion found to be 10 per cent. From this extreme value of 10 per cent for a 15-volt test piece the efficiencies of corrosion for this voltage varied downward, decreasing nearly to zero in many cases. Nos. 31 and 32 of Table II were tested on 5 volts. At the end of 5,300 hours 0.43 and 0.53 ampere hours per sq. in. of anode surface had passed. When broken open there was no sign of corrosion whatever. For test pieces of the same form the various current densities were in practically the same ratio as the voltages at like periods of the tests. The decrease in the efficiency of corrosion therefore does not take place in the same proportion as the decrease in current density. For the type of test pieces used in these tests and for the conditions of the tests it seems that corrosion practically ceases at a voltage somewhat below 15 volts.

An additional experiment which shows the effect on the efficiency of corrosion of varying the voltage and current density was carried out on test pieces from the same group of 2-in. mortar cubes as Nos. 87 and 88 of Table VII. These were tested in the same way as those of Table VII, excepting that the voltages on the different test pieces varied from 1 to 15 volts. Condensed data of the tests are given in Table VIII.

On comparing the results of these tests with those previously described it is seen that while an increase of voltage from 4 to 15 volts in the case of the 2-in. cubes caused an increase in efficiency of corrosion from 1 per cent or a little more, to about 5 per cent, an increase of voltage from 15 volts to 50 volts on the 6 x 8-in. cylinders caused an increase in the efficiency of corrosion from

TABLE VIII.—EFFICIENCY OF COMPARISON WITH VARIOUS CURRENT DENSITIES.

No.	Brand of Cement.	Voltage.	Ampere Hours.	Anode Loss in grams.	Theoretical Loss.	Efficiency of Corrosion, per cent.	Amperes (average) per sq. in.
98	F	1	0.00034	.002	0	0	0
99	"	1	0.00034	.001	0	0	0
100	"	4	1.15	.015	1.2	1.25	.0022
101	"	6	1.74	.031	1.82	1.70	.0034
102	"	9	2.28	.085	2.38	3.50	.0049
103	"	11	1.40	.116	1.46	8.0	.0075
104	"	15	1.57	.084	1.64	5.1	.013

2 or 3 per cent to about 40 per cent. The comparison of these results, together with the fact that where the high efficiencies of corrosion prevailed there was always high temperature, led to other tests which were designed to show the possible effects of temperature on the efficiency of corrosion.

15. EFFECT OF TEMPERATURE ON THE EFFICIENCY OF CORROSION.

In order to carry out the temperature experiments the test pieces were enclosed in metal vessels with wooden tops which could be screwed down tight to prevent the rapid evaporation of water, and placed in an electrically heated bath of transformer oil. To obtain a fair average result for each value of the temperature, current density at the anode surface, etc., the test pieces were in sets of four, the four test pieces being connected in

series so that the current density might be the same in each. Distilled water served as the electrolyte and the metal vessels answered for a cathode.

The first series of tests was carried out using varying current density and constant temperatures. The object of this test was

TABLE IX.—CORROSION TESTS AT 100 DEG. C. WITH VARIOUS CURRENT DENSITIES.

No. of Test Piece.	Time of Test, hours.	Total Current, amperes.	Current Density m. a. cm.^{2}	Ampere Hours.	Loss by Corrosion, grams.	Efficiency of Corrosion, per cent.	Average Efficiency of Corrosion, per cent.
105	148.50	0.010	0.25	1.48	0.586	37.8	40.6
105A	"	"	"	"	0.653	42.1	....
115	"	"	"	"	0.626	40.3	....
111	"	"	"	"	0.657	42.3	....
100	49.00	0.030	0.78	1.47	0.714	46.3	43.7
104	"	"	"	"	0.583	38.0	....
103	"	"	"	"	0.654	42.4	....
102	"	"	"	"	0.743	48.2	....
83	24.00	0.072	2.00	1.728	0.780	43.3	44.3
80	"	"	"	"	0.727	40.0	....
84	"	"	"	"	0.796	44.2	....
97	"	"	"	"	0.861	47.8	....
85	12.60	0.120	3.50	1.51	0.760	48.4	49.2
82	"	"	3.30	"	0.724	46.1	....
79	"	"	3.40	"	0.820	52.2	....
75	"	"	"	"	0.788	50.2	....
96	6.75	0.200	5.12	1.35	0.655	46.3	48.9
86	"	"	5.55	"	0.679	48.1	....
94	"	"	5.40	"	0.715	50.7	....
87	"	"	"	"	0.711	50.5	....
76	7.00	0.265	7.30	1.85	0.786	40.7	43.5
74	"	"	7.70	"	0.886	45.9	....
77	"	"	7.30	"	0.831	43.0	....
92	"	"	"	"	0.860	44.5	....
91	3.66	0.370	9.70	1.35	0.643	45.6	46.5
88	4.00	"	10.60	1.48	0.805	52.2	....
89	"	"	10.30	"	0.742	48.1	....
73	"	"	"	"	0.645	41.8	....
95	6.50	0.350	"	2.27	1.106	46.6	....
93	7.25	"	"	2.53	1.156	43.7	....

to ascertain whether or not any variation in efficiency of corrosion might be directly due to a change in the current density over the surface of the electrode. The only temperature which could be relied on for constancy with all values of I^2R in the test pieces was boiling temperature, which, for the electrolyte, was a trifle

more than 100 deg. C. Seven different current densities were used, the current density for each set of test pieces being kept constant throughout the test. The results obtained are tabulated in Table IX while the curve of Fig. 11 shows the efficiency of corrosion plotted as a function of current density.

It is there seen that increasing the current density in the ratio of 1 to 40 causes no material change in the efficiency of corrosion. For a given current density there are variations in the efficiencies of corrosion between individual test pieces, but these variations may be attributed to variations in the concentrations of the dissolved substances of the electrolyte. A slight

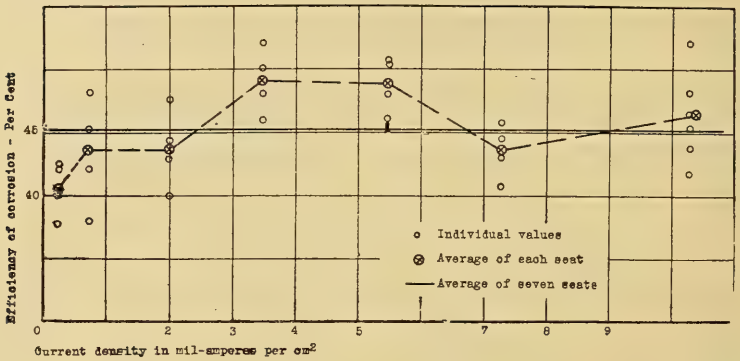


FIG. 11.—EFFICIENCY OF CORROSION IN CONCRETE WITH VARYING CURRENT DENSITY AND A CONSTANT TEMPERATURE OF 100 DEG. C.

variation in the concentration of one of the solvents could easily cause a marked change in the efficiency of corrosion.

A second series of tests, the purpose of which was to show the effect of variation of temperature on the efficiency of corrosion, was conducted in a manner similar to that of the preceding series of tests. The range of temperature was from 2 to 100 deg. C., the temperature of the electrolyte being the one measured and recorded. In order that the heating effect of the current on the interior of the test piece might be nearly the same at all temperatures the current was kept constant and equal to 0.030 ampere through the four test pieces of each set in series. The total average voltage required for this was about 26 volts. The power expended in all four test pieces was, therefore, 0.78 watt, which was

TABLE X.

No. of Test Piece.	Time of Test, hours.	Ampere Hours.	Loss by Corrosion, grams.	Temperature of Electrolyte, degrees Centigrade.	Efficiency of Corrosion, per cent.	Average Efficiency of Corrosion, per cent.
142	51.5	1.54	0.026	3.0	1.60	.....
150	"	"	0.037	"	2.31	.....
146	"	"	0.017	"	1.06	.....
141	"	"	0.012	"	0.75	1.43
99	48.5	1.45	0.069	24.8	4.54	.....
108	"	"	0.044	"	2.90	.....
110	"	"	0.068	"	4.47	.....
113	"	"	0.053	"	3.42	3.84
109	47.8	1.43	0.048	31.0	3.20	.....
107	"	"	0.051	"	3.40	.....
116	"	"	0.041	"	2.73	.....
106	"	"	0.039	"	2.60	2.98
132	48.2	1.45	0.038	39.5	2.51	.....
114	"	"	0.042	"	2.76	.....
101	"	"	0.054	"	3.57	.....
112	"	"	0.046	"	3.04	2.97
138	48.0	1.44	0.031	49.3	2.06	.....
122	"	"	0.058	"	3.86	.....
137	"	"	0.038	"	2.53	.....
139	"	"	0.041	"	2.73	2.79
144	48.0	1.44	0.206	54.5	13.70	.....
135	"	"	0.102	"	6.80	.....
143	"	"	0.285	"	19.00	.....
136	"	"	0.174	"	11.50	12.70
124	48.0	1.44	0.103	59.9	6.86	.....
121	"	"	0.138	"	9.20	.....
123	"	"	0.218	"	14.53	.....
119	"	"	0.102	"	6.80	9.35
125	48.3	1.45	0.384	70.0	25.40	.....
121	"	"	0.343	"	22.70	.....
128	"	"	0.375	"	24.80	.....
118	"	"	0.322	"	21.30	23.50
140	48.0	1.44	0.692	79.8	46.10	.....
126	"	"	0.479	"	32.00	.....
129	"	"	0.702	"	46.80	41.60
130	"	"	0.740	89.5	49.30	.....
134	"	"	0.733	"	48.80	.....
133	"	"	0.763	"	50.80	49.60
102	49.0	1.47	0.743	100.0	48.30	.....
103	....	....	0.654	.....	42.40	.....
104	....	....	0.583	.....	38.00	43.70
100	....	....	0.714	.....	46.30	.....

not sufficient to cause any appreciable rise of temperature. The condensed data of the tests are given in Table X. Fig. 12 shows a plot of efficiency of corrosion as a function of temperature, the temperature of the electrolyte being the one used as mentioned above.

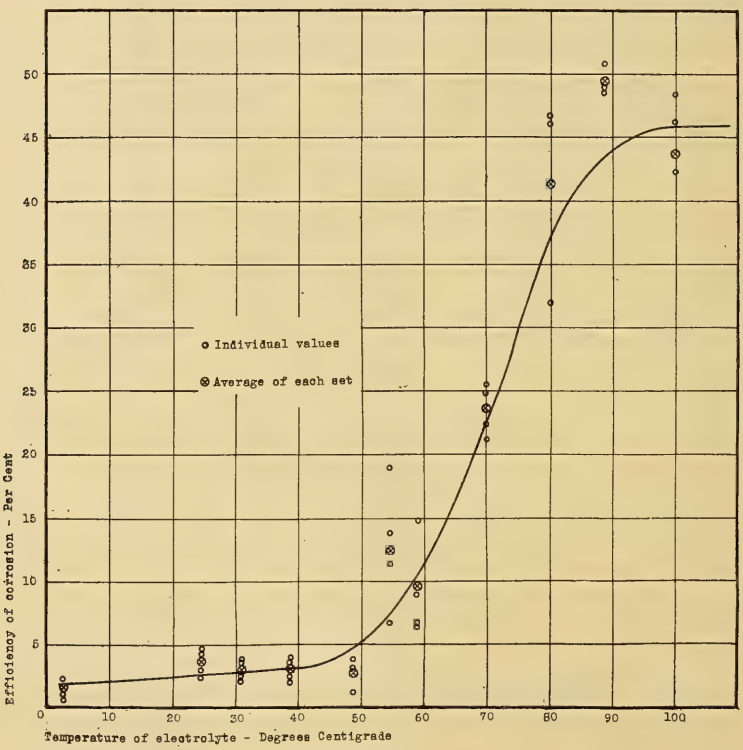


FIG. 12.—EFFICIENCY OF CORROSION IN CONCRETE AS A FUNCTION OF TEMPERATURE.

The most prominent characteristic of the curve of Fig. 12 is the sharp bend upward which occurs in the neighborhood of the temperature of 50 deg. Below 50 deg. the efficiency of corrosion is very small. Since current density has been shown to have no direct effect on efficiency of corrosion the great difference in efficiency of corrosion between high and low voltage test pieces must therefore be attributed to the heating effect of the current.

If the heating effect of the current is sufficient to raise the internal temperatures of the test pieces to 50 deg. C. or more, active corrosion begins. Below 50 deg. the iron remains passive, or nearly so. This has been found to be strictly true, however, only for normal concrete test pieces to which no foreign ingredient of marked corrosive tendencies has been added as will presently appear.

16. EFFECT ON THE EFFICIENCY OF CORROSION OF ADDING SALTS TO CONCRETE.

The results of low voltage tests on concrete test pieces to which a small percentage of salt (NaCl) had been added showed that passivity of iron in concrete is thereby destroyed almost completely for all voltage and conditions of temperature. Calcium chloride has the same effect, the chlorine ion being the one which affects the iron. In one case it was found that the addition of 0.02 per cent, by weight of the cement, of $CaCl_2$ was sufficient to cause a very marked increase in the efficiency of corrosion, while 0.33 per cent caused the efficiency of corrosion to rise to 80 per cent.

17. EFFECT OF AGING OF CONCRETE ON EFFICIENCY OF CORROSION.

Since passivity of iron in normal concrete is brought about largely by the presence of calcium hydroxide in solution in the pores of the concrete the question arises as to whether the low values of the efficiency of corrosion obtained with low voltages in comparatively new concrete would still be obtained after the concrete is several years old. There is a possibility that after the $Ca(OH)_2$ has become thoroughly carbonated the passivity of iron may be destroyed for all voltages and temperature conditions. Tests are in progress relating to this phase of the matter but no results are available at the present time.

18. USE OF WATERPROOFING MATERIALS.

The effect on the efficiency of corrosion by adding various waterproofing agents to the cement was also tested by a considerable number of experiments with the result that in most cases the results were substantially the same as with untreated test pieces. Where there was a difference the effect of the added ingredient was usually to *increase* the *corrosion.*

Attempts to keep moisture out of concrete with a view of increasing its resistance and therefore reducing the current flow under a given voltage have been generally unsuccessful. In this connection there have been tested a great many so-called waterproofing agents consisting of substances that are to be added to the water, the cement, or to the aggregate while mixing. It will not be attempted to give the details of all these tests here, but merely to state that practically all of the materials tested have been found to be of practically no value in reducing current flow, at least after the first few weeks. By the application of waterproof membranes, etc., to the surface, much more promising results were obtained, and it seems altogether probable that these may be made to have considerable protective value. The experiments on which these conclusions are based are described in detail in *Technologic Paper* No. 13 of the Bureau of Standards.

III. POSSIBILITIES OF TROUBLE FROM ELECTROLYSIS IN CONCRETE STRUCTURES UNDER PRACTICAL CONDITIONS.

As mentioned in the introduction, reports have become current from time to time during the last few years that more or less serious trouble has occurred in concrete structures as a result of electrolysis and in some cases damage amounting to practically complete destruction has been reported. The consequences that might possibly arise from such damage are obviously very serious, and since the reports of such trouble have in some instances been made in leading technical papers by reputable engineers, it becomes imperative that a thorough investigation of the practical aspect of the problem be made. In connection with the investigations described in the preceding sections, every effort has been made to study the matter from the practical standpoint also, in order to determine, as far as possible, to what extent the conditions under which concrete can be injured in the laboratory, may be expected to obtain in practice.

19. CONDITIONS NECESSARY FOR DAMAGE TO OCCUR.

A careful study of the data presented in the preceding chapters shows quite conclusively that while there are conditions under which reinforced concrete may be seriously injured such conditions

are nevertheless decidedly exceptional rather than the rule. These exceptional conditions do occur, however, with sufficient frequency to make the problem one of great importance and fortunately most of these conditions are amenable to control. It has been seen that the most important essentials in the production of unsound concrete by electrolysis are moisture and a difference of potential between electrodes in contact with the mass of the concrete. At first thought it might appear that these two conditions are almost omnipresent, since perfectly dry concrete, especially below grade, is seldom if ever found; while, as every electrical engineer knows, there are few places in our cities at the present time where appreciable differences of potential cannot be found between any two points more than a few yards apart. The statement in regard to the rarity of dry concrete is made advisedly, since only the most minute quantities of moisture are necessary in order to impart to concrete a very considerable conductivity. On the other hand, a state of practical saturation is necessary in order to impart to the concrete a maximum of conductivity and any reduction of the moisture content below the saturation point causes an increase in its resistance, and a consequent decrease in the current which will flow through the concrete under a given potential gradient. As shown by data already presented, the resistance of ordinary air-dried concrete is of the order of about ten times that of wet concrete, and for this reason concrete above grade is much less susceptible to electrolytic damage than if so located as to be permanently wet. It is not to be inferred, however, that air-dried concrete is immune from electrolysis troubles, but merely that a much higher voltage is nec ssary in order to produce trouble; and in general in the absence of special conditions to be mentioned in the next section, electrolytic damage to concrete above grade will be extremely rare.

The condition mentioned above, that the electric current must flow between electrodes in contact with the concrete, cannot be too strongly emphasized. The conduction being electrolytic, the reactions take place only at the electrodes, and in the absence of such electrodes no reactions occur within the concrete. The only effect, therefore, would be the slow removal of the water soluble constituents and hence the effect on the concrete would not be essentially different from that of s`ow water seepage.

20. SOURCES OF STRAY CURRENTS.

If there be electrodes imbedded within the concrete, as in the case of reinforced concrete structures, the electrode effects described in the foregoing section may be expected, provided the voltage is sufficient. The sources of potential differences in concrete structures may be classed under two heads, (1) those due to direct contact between the conductors of lighting or power circuits, and some part of the building and (2) those which have their origin in stray currents from railways or other grounded power lines. The former may happen in any building containing electric wires, through defective insulation. It is not necessary, of course, that both sides of the line be grounded in the building itself, since, if one side of the line is grounded on the building and the other grounded in some remote quarter of the system, those portions of the building itself near the grounded wire will be subjected to a considerable difference of potential. If the wire be grounded directly on the concrete and not on the reinforcement the comparatively small cross-section of the path of the current near the point of contact between the concrete and the wire will result in most of the total drop of potential to ground occurring within a restricted region near the wire and it is only here that any damage may be expected, and since the current will be small the damage if any will be small. Ultimately, of course, any current that leaks off from the wire would pass out into the earth through the footings and foundations, and through pipe systems entering the building. As a rule the cross-section of these paths is so large in the aggregate, that the potential gradients would not be sufficient to raise the temperature appreciably, and hence no appreciable damage can occur. If the current be reversed, flowing to the building from outside, there would in time be some softening of the concrete in a thin layer under and around the steel structure terminating in the footing, but this would be under compression and not subjected to shear, so that failure here is extremely improbable. The only places where trouble is to be expected due to grounding of power wires directly on the concrete inside of a building, is in the region close to the point of ground.

If, however, the power wire be grounded directly on a portion of the reinforcing material, the condition will be more serious,

and the extent of the danger will be greater if there is a large quantity of the reinforcing material in metallic contact with the electric circuit. If this comprises a large part of the total reinforcement of the building the condition might be serious, irrespective of whether the positive or negative side of the line is grounded. If the ground is on the positive side the potential gradient near the reinforcement may become high enough to cause rapid corrosion and consequent destruction of the reinforcing material. If on the other hand the reinforcing material be negative there would develop a softened condition of the concrete near the surface of the iron which would practically destroy the bond, and this would probably be the more serious condition of the two since the latter would not manifest itself by producing local cracks in the concrete, and might not become known until a large portion of the building has become weakened. However, while such a condition as this might occur, and if neglected become very serious, it is nevertheless a trouble that can be readily guarded against as will be pointed out below.

The other source of current that might possibly give rise to trouble under certain circumstances is the ground return of railways. The current may enter a building in two different ways. First, if the foundations under the two opposite sides of the building are at different potentials there would be a tendency for a certain amount of current to flow up through the foundation on the one side, through the walls and floors of the building, and out through the foundation on the other side. This condition may be said to exist to a very small extent in practically all concrete buildings, but it is not one that need cause any alarm. In the course of numerous electrolysis surveys carried out in various cities it was found that a potential difference exceeding a few volts due to stray currents between any two parts of a building is extremely rare, and this would almost inevitably be distributed over so great a distance that the potential gradient would not be sufficient to cause any appreciable trouble, in view of the experimental results set forth in the first section of this paper, which show that a potential gradient of at least several volts per inch is required before any damage results.

The second way in which stray currents may enter a building is through water or gas pipes, lead cable sheaths and similar

structures. In this case considerably larger differences of potential may be brought about between different portions of the building and between parts of the building and the earth. If the pipe systems come in contact only with the concrete and not with the reinforcing material, any damage that may occur will be slight and will be confined chiefly to the immediate vicinity of the pipes or cables; but if the pipes come into metallic contact with the reinforcing material, the latter comes to the same potential as the pipes and may become either anode or cathode according to the condition of the pipes. Cases may arise in practice where differences of potential of serious magnitude may be produced in this way, some instances having been noted in which the reinforcing material was from 5 to 15 volts above or below the earth. Under most conditions damage would proceed very slowly under such differences of potential as these, but, nevertheless, wherever voltages of this magnitude exist it should be regarded as a dangerous condition and should be remedied at once.

21. INCREASED DANGER DUE TO PRESENCE OF SALT.

The above statements in regard to the liability of damage under low or moderate differences of potential are intended to apply only to concrete which contains no appreciable quantities of salt. The data given in the preceding section show that if a small quantity of sodium chloride or calcium chloride be added to the concrete the rate of deterioration proceeds many times faster, and under such circumstances much lower voltages should be regarded as dangerous. Even a small fraction of one per cent. of chlorine in concrete is capable of increasing the electrolytic corrosion of the iron many fold, and if salt has been added to the concrete during construction, or if it comes into contact with salt water afterward, much greater precautions are necessary in order to prevent damage.

In the course of these investigations there have been examined a considerable number of cases in which damage to concrete structures has been attributed to electric currents. Some of these have been reinforced structures and some have been without reinforcement. Among these there have not been found any non-reinforced structures in which the conditions indicated that

electric currents could in any way be responsible for the damage. Among the reinforced structures which have been noted there are some in which electrolysis has been at least a contributing cause of the damage. There has not, however, been any case in which serious damage has occurred in which there was not also present a considerable quantity of salt in the concrete, either from having been put there during construction, or from contact with salt water in service. This is in accord with the results of the laboratory investigations, which show that under low or moderate voltages, the rate of damage, in the absence of chlorine, is so slow as to be almost negligible, but that when salt is present rapid deterioration at both electrodes may be expected even on comparatively low voltage.

22. SOME SPECIFIC CASES OF TROUBLE.

In making an examination of these cases the condition of the concrete was carefully noted and compared with that of specimens known to have been injured by electric currents in the laboratory. Potential measurements were made between different parts of the structure, and also between the structure itself and surrounding structures, although the latter are as a rule not important since the electrical resistance between different structures is usually unknown, and hence the voltage readings give no idea of the current flow. By comparing the potential gradients observed with those used in the laboratory and considering also the age of the structure and any special conditions which may have influenced it, we are enabled to judge as to whether it is possible for the damage to have been caused by electric currents, and this judgment is further confirmed by studying the character of the damage that can be caused by electrolysis. One or two specific cases may be mentioned here as instances.

The concrete foundations of a small bridge across the Gowanno Canal at Hamilton Ave., South Brooklyn, developed large cracks in a number of places, and the damage had been attributed by some to electrolysis. On the strength of the first reports the Bureau was inclined to attribute the trouble to electrolysis, since a trolley line passed over the bridge and it seemed quite probable that conditions might have arisen whereby a sufficiently high

potential gradient had been produced to corrode the reinforcing material and crack the concrete. On making an examination of the bridge, however, it was found that those portions in which failure had occurred did not contain reinforcing material, and the cracks were not so located that they could have been caused by corrosion of iron, nor could there have been a weakness due to a cathode effect, since there were no electrodes near the cracks, and the concrete nowhere showed evidence of softening. In short, there was no similarity between the damage found here and any of the effects which had been noted in the laboratory. Further, potential measurements showed that the maximum potential difference to be found was 1.6 volts, while potential gradient measurements between points two feet apart near some of the cracks showed values ranging from 0.01 to 0.05 volts per foot. In the experiments of the Bureau of Standards there have been impressed on wet concrete blocks potential gradients of about 100 volts per foot, continuously for a year and a half using non-corrodable electrodes, without producing any cracking such as observed in this bridge. It seems inconceivable that such small potential gradients as found here could have been responsible for the cracking of the unreinforced portions of the structure, when potential gradients in the laboratory 2,000 times greater failed to produce any damage at all, except such as noted in the foregoing chapters and which was always closely associated with the reinforcement in the concrete. Further, there was evidence that the foundations of the structure were unstable as shown by the fact that they had moved several inches toward the canal since they were built. Somewhat similar trouble has been experienced in a number of buildings in this vicinity, indicating that the earth in this locality does not afford a stable support for the foundations. This alone would seem to be capable of causing trouble of the nature that has actually occurred here, whereas as first indicated the damage is not of a nature that has been observed to result from the passage of electric current.

One other instance may be cited here because of its importance. Some concrete lined railway tunnels had suffered considerable damage due to disintegration of the concrete, and in some instances within a few months after work had been completed. A number of engineers had pronounced the trouble due to electrol-

ysis and the engineers of the Bureau of Standards were asked to make an investigation. A voltage survey throughout the tunnels was made, potential measurements being taken between points on the tunnel walls in both a vertical and horizontal direction, and in some cases holes were drilled in the wall and voltage measurements made in a direction at right angles to the surface. Some measurements were also made between walls on opposite sides of the tunnel, and in two or three cases between the tunnel wall and the rails. The measurements taken were classified under three heads, viz: those taken where the concrete was very badly disintegrated, those taken where there was but slight disintegration, and those taken where the concrete was perfectly sound. A comparison of the magnitudes of the voltages showed first that there is no definite relationship between the magnitude of the potential gradients observed and the physical condition of the concrete. The average voltage per foot in the first group where the concrete was very badly damaged was 0.017 volt. In the second group where there was but slight damage, there was an average of 0.011 volt per foot, while in the third group comprising those places where there was no damage whatever the average potential gradient was 0.027 volt per foot. Thesefi gures do not support the view that the trouble is caused by stray electric currents. It should be noted that in a number of places measurements were obtained where the concrete was very wet and yet in perfect condition, and in which the potential gradients were of the same order as those in the first group where the concrete was badly damaged.

In taking the readings the usual method of making contact at points between which measurements were to be taken could not be used, because the high resistances at the contact points would absorb the major part of the total voltage. Special terminals were designed consisting of brass cups 2 in. in diameter in each of which was compressed a sponge saturated with a solution of copper sulphate. Inside the cup and below the sponge was a brass piston which was used as the instrument terminal. When readings were to be taken the cups were placed against the concrete wall at points between which the voltage was to be measured with the piston pressing the sponge firmly against the wall. This gave a large surface of contact and therefore a relatively low

resistance. Even in this case however, it was found that the resistance of the contacts was still too high to permit of using a voltmeter, so a special high sensibility portable galvanometer connected in series with a megohm resistance was used. Careful tests in the laboratory showed that the accuracy of the measurements was within three per cent, even on nearly dry concrete, and considerably higher when the concrete was wet.

The fact that there is no definite relationship between the voltages observed and the condition of the concrete is not surprising, particularly in view of the low values of most of the readings. In the experiments at the Bureau of Standards blocks of wet concrete have been subjected to potential gradients varying from very low values up to several thousand times the average values observed in the tunnels mentioned above, for a period of one and one-half years, without producing any appreciable diminution in the crushing strength of the main body of the concrete, and there is no evidence of any other action except in the immediate vicinity of the electrodes. It is desired to emphasize the fact that in concrete, as in all other electrolytic conductors, all chemical reactions which take place as a result of the current are essentially electrode effects, and they do not, except by progressive action from the electrodes, affect the mass of the concrete.

In the region of the cathode, there is a softening of the concrete, but, as pointed out above, it begins as a thin film on the surface of the cathode and very slowly progresses outward. There is a sharp line of demarkation between the injured and uninjured area, the main body of the concrete being unaffected. The rate at which this softened region progresses outward from the electrode is so slow that it could not possibly give rise to the phenomena observed in the tunnels in question. Experiments with normal concrete under potential gradients up to about 5,000 times those found in these structures show that the rate at which the softened region moves outward from the cathode is not over about an inch per year, and the concrete outside this very restricted area always remains perfectly sound. In the light of these experiments and considering the relatively low potential gradients existing in and around these concrete structures, the theory that damage of this nature could have spread over such vast areas in so short a time is untenable. After a careful study of the conditions existing

in these tunnels, the Bureau was forced to the conclusion that electrolysis has played no appreciable part in the damage which has been experienced.

23. PROCEDURE IN MAKING VOLTAGE MEASUREMENTS ON CONCRETE STRUCTURES.

It is desired to say a word here in regard to the proper method of taking electrical measurements that are intended to show whether or not a structure is being affected by stray currents. The apparatus which has been found successful has already been described. It is not alone important to be able to read the voltages correctly, but it is equally important to take the measurements between the proper points and to properly interpret them. Cases are known where damage has been attributed to stray currents on the strength of potential measurements taken between the structure and the rails of a near by steam railroad. The rails near the structure, and for a half mile or so on either side, were supported out of contact with the soil on wooden ties so that the potential measurement that was being taken was between structure and a point several thousand feet distant. It seems almost needless to say that a potential measurement of this kind is worse than worthless as an indication of the electrical condition of the structure under examination. To get an idea of the extent to which currents may be flowing in a building by means of potential measurements it is necessary to take the potential readings between two points within or on the building itself and not between the building and some other structure more or less insulated from it. Similarly to determine whether current is flowing into or out of a building the potential measurement should be made between points on the building and thoroughly grounded points near by. Failure to keep these facts in view appears to have been responsible for a good many erroneous reports of damage to concrete structures by stray currents.

IV. PROTECTIVE MEASURES.

24. EXCLUSION OF SALT.

In considering measures that may be taken to cure electrolysis troubles, or prevent possible damage, it is not necessary to consider structures which have no metal imbedded in them, since, as

pointed out above, these are immune from electrolysis effects. As regards reinforced concrete buildings (or concrete buildings with metal conduits imbedded in the concrete), there are two cases to consider: (1) new structures in process of building, and (2) structures already completed. In the case of new structures of reinforced concrete salt should be omitted altogether, if there is the slightest probability that they may ever be subjected to the action of electric currents. Calcium chloride is quite as bad as sodium chloride, it being the chlorine that does the harm. Since the addition of even a fraction of one per cent of chlorine is sufficient to increase the rate of damage a hundred fold, it is impossible to use a sufficient quantity of salt to lower the freezing point of concrete perceptibly without making the structure decidedly more vulnerable to the action of electric currents.

25. WATERPROOFING BELOW GRADE.

A good deal can be accomplished also by proper construction of the basement, foundations and footings of the building. If beneath the foundations and outside the basement walls a layer of insulating material be placed it will prevent the access of electric currents through the building foundations. While such insulating materials can readily be made that will give a permanent increase of resistance, there is yet one to appear that will, in the presence of water, remain completely insulating over a period of years. The Bureau has tested a great many waterproofing membranes but, in the presence of water, all have sooner or later begun to acquire considerable conductivity and show gradual deterioration. It is, of course, not necessary that such insulation should be perfect in order to be of value, since any considerable increase in the resistance which it may produce in the path of current flow will be useful. For this reason, a waterproofing medium, such, for instance, as one made up of multiple layers of fabric treated with pitch, will have sufficient insulating value to be quite effective in preventing the passage of electric currents, and would probably be of considerable value even after the waterproofing has ceased to be perfect.

26. ADDITION OF WATERPROOFING COMPOUNDS.

Attempts to increase the resistance of concrete by the addition of so-called waterproofing compounds, to the cement before mixing,

have uniformly failed to give satisfactory results. As pointed out in the first section of this report, none of these compounds or mixtures that have been brought to the authors' attention are of great value in this connection, and reliance should not be placed on them as a means of protecting structures against electrolysis.

27. CONSTRUCTION OF FOUNDATIONS.

A good deal can be done in the way of increasing the resistance between the building and ground by a proper selection of materials for the foundations. Blocks of granite, frequently used in such work, have a much higher resistance than concrete, a number of specimens tested showing, in the water-soaked condition, a resistance approximately one hundred times as great as that of wet concrete, and hence if granite blocks be interposed between the footings and the soil the tendency of the building to pick up stray currents will be greatly reduced. This matter is regarded as in most cases of secondary importance if the potential differences in the region of the building are as small as usual, since any currents which a building may pick up this way will be too small to do any appreciable harm. In some cases, however, the use of granite footings may be justified as a measure of precaution.

The foregoing statements are not applicable to structures already built, but the following preventive measures may as a rule be applied to old as well as new structures.

28. ELECTRIC WIRING.

In wiring a concrete building with direct current circuits it is of the greatest importance that the construction be such as to preclude the possibility of either side of the circuit coming in contact with the concrete. The insulation should be of the best grade and the wires should be enclosed in continuous metal conduits. Periodic tests should be made on the insulation and any defect remedied. If the power supply is a private plant belonging to the building the installation of ground detectors is desirable. If these precautions are taken it will not be necessary to insulate the conduits from the building.

29. INSULATION OF PIPES AND CABLES.

By far the most important path of entry of electric currents into concrete structures from outside sources is through the pipes

and lead cable sheaths, etc., which enter the building, and it is only through these paths, as a rule, that sufficiently high potential differences can be produced to result in damage to the structure. The most effective remedy therefore, and, at the same time, a comparatively simple one, is to introduce insulating joints into the pipes before they enter the building. Such joints can easily be made, and are in fact now used in many places, although not, so far as the authors are aware, in this particular connection. The only condition to guard against is the possibility of developing too high a difference of potential across the joint, which might give rise to electrolytic injury to the pipes; but this can be obviated by proper construction of the joint, as by giving it a long leakage path and by putting several insulating joints in series.

When a pipe line passes through a building it will be necessary, if insulating joints are used at all, to use them on both sides of the building. To use them on one side only would result in making the building more strongly positive or negative to the earth than it would be without such joints and thus the danger would be increased. If the insulating joints are put on both sides, however, there will be no likelihood of currents entering the structure to any appreciable extent, except in special cases where the difference of potential between the broken sections of pipe becomes very high, say of the order of 10 or 15 volts or more. When this condition occurs, however, it can be alleviated by shunting around the insulated section of pipe within the building by means of a copper cable, preferably an insulated one.

It was stated above that lead covered cables also form a possible source of entry of stray currents into buildings. It is, of course, possible to break up the continuity of lead cable sheaths by inserting insulating joints, but there are more serious objections to doing this than in the case of water or gas pipes. Nevertheless in some instances this is to be recommended; but it would usually be better simply to insulate the cable from the building, since only a very low degree of insulation would be necessary, such as would be obtained by carrying the cable on wooden supports and keeping it out of actual contact with the concrete and reinforcing material.

30. MAKING THE REINFORCING MATERIAL NEGATIVE.

A method of protecting reinforced concrete structures that has frequently been proposed, viz: by making the reinforcing

material continuous throughout the building and connecting it to the negative terminal of a low voltage generator, should be referred to here, but only to point out the danger attending it. While there is no question but that corrosion of the reinforcing material might be prevented in this way, the experiments described in the first section of this paper show that under ordinary low voltages there is greater danger in having the iron negative than positive on account of the destruction of the bond in the former case. Any condition, therefore, which may cause the reinforcing material to become negative to the concrete should be carefully avoided.

31. GROUNDING OF METALLIC CONDUITS.

The practice at present followed in many instances, and made compulsory in some city ordinances, of grounding all metallic conduits in contact with the concrete is not to be recommended as a general rule. There are, of course, many cases under which such grounding would not bring about a condition dangerous to the concrete, but in some cases at least the results of such grounding might be serious. If the ground were made on the water pipes as is commonly done, and if the district were one in which the pipes were at a higher or lower potential than the earth, the conduits would be made either anode or cathode, a condition which, in general, should be regarded as unsafe, and this condition would in fact be worse than the unrestricted entry of pipe systems into the buildings, which should be guarded against as pointed out above. If, on the other hand, the ground is made through a ground plate under or near the building, this will bring all of the conduit system to the same potential as the ground plate, and this being of small area compared to that covered by the building, the footings of the building remote from the ground plate may show a considerable difference of potential against the ground plate, and consequently against the conduit of the building, thus giving rise to flow of current between the conduit and the structure itself. If the low voltage side of alternating current circuits be grounded at the neutral point, there is no material advantage to be gained by grounding the conduit, and in the case of metallic conduit embedded in concrete, the authors would advise against such grounding.

32. IMPROVING THE NEGATIVE RETURN OF RAILWAYS.

In discussing means of preventing damage to buildings due to stray currents, sight should not be lost of the fact that, after all, the most effective way of reducing damage due to stray currents is by removing, as far as practicable, the cause of such currents. This means a proper design, construction and maintenance of the railway distribution system, whereby potential gradients in the earth are not permitted to reach such high values as to give rise to serious trouble. The protective measures against stray currents mentioned above are rather to be regarded as precautionary ones, rendered necessary only on the failure of the railway companies to provide adequate means for returning their currents to their source. If the negative return systems of the electric railways of this country were uniformly as good as those maintained in the principal countries abroad, the problem of damage to buildings by stray currents would practically disappear.

V. CONCLUSIONS.

The following conclusions are drawn from the investigations herein above described:

1. The observations of previous investigators that the passage of current from an iron anode into normal concrete caused the destruction of the test piece by cracking the concrete were only partly confirmed. This effect was found not to occur in most of the test pieces when the potential gradient was less than about 15 volts over a distance of 3 in. or about 60 volts per foot.

2. Of the numerous theories that have been advanced for the cracking, that one which attributes it to oxidation of the iron anode following electrolytic corrosion has been fully established. The oxides formed occupy 2.2 times as great a volume as the original iron, and the pressure resulting from this increase of volume causes the block to crack open.

3. Metals which do not form insoluble end products of corrosion and all non-corrodable anodes, never cause cracking of the concrete as a result of the passage of an electric current.

4. The mechanical pressure developed at the iron anode surface by corrosion of the iron has been measured in a number of cases and has been found to reach values as high as 4,700 lbs. per

sq. in., a value more than sufficient to account for the phenomena of cracking that have been observed.

5. Suggestions of some engineers that copper-clad steel or aluminum be used as reinforcing material have been shown to be impracticable, since the copper coating is readily destroyed and the aluminum is attacked by the alkali in the concrete.

6. Corrosion of iron anodes in normal concrete is very slight at temperatures below about 50 deg. C.

7. For any fixed temperature the amount of corrosion for a given number of ampere hours is independent of the current strength.

8. The lack of corrosion of the iron at temperatures below 45 deg. C. is due to the inhibiting effect of the $Ca(OH)_2$ in the concrete.

9. The rapid destruction of anode test pieces at high voltages (60 to 100 volts or more) is made possible mainly by the heating effect of the current, which raises the temperature above the limit mentioned above. If the test piece be artificially cooled no appreciable corrosion occurs, and no cracking results.

10. The potential gradient necessary to produce a temperature rise to 50 deg. C. with consequent corrosion, in the test pieces used, was about 60 volts per foot. For air dried concretes it is much higher. This shows that under actual conditions corrosion from stray currents may be expected only under special or extreme conditions as noted below.

11. Since the passivity of iron in concrete is due chiefly to the $Ca(OH)_2$ present, it appears probable that old structures in which the $Ca(OH)_2$ has been largely converted into carbonate will be more susceptible to the effects of electric currents than comparatively new concrete with which the foregoing experiments have been made. The increase in the efficiency of corrosion would, however, be at least partly offset by the increase in the resistance of the concrete which would accompany the change.

12. The addition of a small amount of salt (a fraction of one per cent) to concrete (as is frequently done to prevent freezing while setting) has a two fold effect, viz: it greatly increases the initial conductivity of the concrete, thus allowing more current to flow, and it also destroys the passive condition of the iron at ordinary temperatures, thus multiplying by many hundreds of

times the rate of corrosion and consequent deterioration of the concrete. Salt should, therefore, never be used in structures that may be subjected to electrolytic action. Further, reinforced concrete structures built in contact with sea water, or in salt marshes, are very susceptible to electrolysis troubles.

13. Test pieces of normal concrete carrying currents increase their resistance a hundred fold or more in the course of a few weeks, which fact still further lessens danger of troubles.

14. The rise of resistance is in general due to precipitation of $CaCO_3$ within the pores of the concrete, thus plugging them up. A slight amount of salt prevents this precipitation and consequent rise of resistance, thus still further emphasizing the detrimental effect of salt.

15. Contrary to the observations of previous investigators there is a distinct softening of the concrete near the cathode. This begins at the cathode surface and slowly spreads outward, in some cases as far as $\frac{1}{4}$ in. or more. After exposure to the air this softened layer becomes very hard again, but remains brittle and friable.

16. The softening effect at the cathode noted above causes practically complete destruction of the bond between reinforcing material and the concrete, reducing it to a few per cent. of its normal value.

17. Unlike the anode effect which becomes serious in normal concrete only on comparatively high voltages, the cathode effect develops at all voltages, the rate being roughly proportional to the voltage in a given test piece.

18. In general the cathode effect occurs under conditions which may frequently occur in practice and is therefore a more serious matter practically than the anode effect about which so much has been heard.

19. The softening of the concrete at the cathode is due chiefly to the gradual concentration of Na and K near the cathode by the passage of electric current. In time the alkali becomes so strong as to attack the cement.

20. Softening at the cathode is increased by increasing the Na and K content of the cement, and reduced by diminishing this content.

21. The softening of the concrete has never been observed,

except very close to the cathode, the main body of the concrete remaining perfectly sound. Numerous tests show conclusively that the crushing strength of the main body of the concrete is not reduced, even when the potential gradient is maintained at 175 volts per foot for over a year.

22. Because of the cathode effect noted above, the method of protecting reinforced concrete buildings by maintaining the reinforcing material cathode as by a battery or booster would be much more dangerous than no protection at all.

23. The only effect which an electric current has on unreinforced concrete is to cause a migration of the water soluble elements. Consequently, in the absence of electrodes, the ultimate effect of current flow on the physical properties of the concrete is not materially different from that of slow water seepage, which also removes the water soluble elements. Non-reinforced concrete buildings are therefore immune from trouble due to stray earth currents. They might, however, be injured by the grounding of power wires carried in conduits embedded in the concrete, since these conduits would then act as electrodes.

24. Conditions do arise in practice which will give rise to damage due to stray currents, but the danger from this source has been greatly overestimated in many quarters. While precautions are necessary under certain conditions, there is no cause for widespread alarm.

25. Waterproofing reinforced concrete would greatly increase its resistance and diminish accordingly the danger from either the anode or cathode effects.

26. Waterproofing to prevent electrolysis is, however, a much more difficult matter than waterproofing to maintain a moderate degree of dryness, because of the much higher degree of waterproofing required in the former case.

27. It has been found that practically all of the waterproofing agents now on the market that are intended to be mixed with the concrete, are of little value as preventives of electrolysis. Waterproofing membranes, etc., applied to the surface can be made much more effective and when properly applied may have considerable effect in preventing the entry of earth currents into the concrete.

28. Painting or otherwise coating iron with an alkali resisting

metal preservative before embedding it in concrete may serve to minimize the dangers of electrolysis, but no such coating has been found that does not prevent the formation of the bond between the concrete and iron when the concrete sets.

29. In order to insure safety from electrolysis, potential gradients must be kept much lower in structures exposed to the action of salt waters, pickling baths, and all solutions of chlorides, sulphates, nitrates, or carbonates.

30. All electric power circuits within the building should be kept free from grounds directly on a portion of the building itself. If the power supply comes from a central station the local circuits should be periodically disconnected and tested for grounds and incipient defects in the insulation. In the case of isolated plants ground detectors should be installed and the system kept free from grounds at all times.

31. All pipe lines entering concrete buildings should, if possible, be provided with insulating joints outside the building. If a pipe line passes through a building and continues beyond, one or more insulating joints should be placed on both sides of the building. If the potential drop around the insulated section is large, say 8 or 10 volts or more, the insulated portion should be shunted by means of a copper cable.

32. Lead covered cables entering such buildings should be insulated from the concrete. Wooden or other non-metallic supports which prevent actual contact between the cable and the concrete will give sufficient insulation for this purpose. Such insulation of the lead covered cable is desirable for the protection of the cable as well as the building.

33. The grounding of electric conduits to water pipes and ground plates is in general not to be recommended in the case of concrete structures.

34. In making a diagnosis of the cause of damage in any particular case, the fact that a fairly large voltage reading may be obtained somewhere about the structure, should not be taken as sufficient evidence that the trouble is due to electrolysis. The distance between the points, and particularly the character of the intervening medium, are of much greater importance than the mere magnitude of the voltage reading. As a precautionary measure, however, all potential readings about a reinforced concrete structure should be kept as low as possible.

DISCUSSION.

Mr. Willis Whited.—I would like to ask if any of the tests indicate whether the composition of the aggregate has any influence on these phenomena. Mr. Whited.

Mr. Burton McCullom.—Only to a comparatively slight extent, unless some foreign ingredient such as a waterproofing agent has been added. The aggregate does have an effect on the resistance, but it is so small as not to be a very important consideration. We have found that certain so-called waterproofing ingredients added to the concrete greatly increased the corrosion, so unless it is known not to produce that effect it is a little risky to use them. Mr. McCullom.

Mr. Cloyd M. Chapman.—How long were the experiments continued that resulted in the reduction of the bond to one-fifth of its original amount? Also, was the softening of the concrete adjacent to the reinforcing material a progressive phenomenon, *i. e.* did those broken at an early period show a small area or depth of softening and those broken at a later period a greater? Mr. Chapman.

Mr. McCollum.—About seven months. It seems to be a progressive action, just about proportional to the amount of electricity that was passed through the test piece. The examinations made on those that had been in circuit varied from a month to a year, and the thickness of the softened part was about proportional to the time. Mr. McCollum.

Mr. Chapman.—The paper states that a large number of blocks subjected to the low voltage for a long time did not crack, and infers that they probably never would crack if that voltage were continued indefinitely. We have made a large number of such tests and have not yet failed to break a block by continuing it long enough. We use a range of 5 to 10 volts, the resistance varying considerably. We cannot maintain it because we do not give it constant attention, but we check it up every few days, alter the resistance to keep it between 5 and 10 volts. We have yet to test a case where the block did not in time break. Our blocks have considerably less area. A number of cubes up to the limit of the experiment had not broken. Mr. Chapman.

Mr. Chapman. Another point, the protection afforded by waterproofing materials. A series of experiments is in progress with concrete made with waterproofing materials, and three blocks, in circuit about two months, have not yet broken, while a blank or untreated block broke in 11 days. One ran three times as long as the untreated block. It is probably quite true there are many waterproofings having the opposite effect, that would accelerate the corrosion and the blocks in which they were used would break sooner than the untreated blocks; but it looks as though there are some waterproofings that can be applied to blocks and the latter will stand longer than the untreated ones.

Mr. Wig. MR. R. J. WIG.—I would like to ask Mr. Chapman what was the longest period required for breaking the blocks on the low voltages he used and as to the size of the block?

Mr. Chapman. MR. CHAPMAN.—It was not over 2 or 3 months with direct current. The blocks were square, 6 in. wide, and the electrode a steel plate, 2 in. wide. The size of the electrode imbedded was about 2 by 5 in. The plates were $\frac{1}{16}$ in. thick. These tests were made for the purpose of testing structural steel paints and coatings and this is the reason plates instead of rods were used for electrodes.

Mr. Auryansen. MR. F. AURYANSEN.—The speaker made mention of the influence of salt. A case was just brought to my attention 2 or 3 days ago of a small lightning arrester that was made of stucco on expanded metal lath. It is over on Jamaica Bay, down near Rockaway Beach, built about seven years ago and is all going to pieces. What is the reason? Is it the salt there or just plain water?

Mr. Chapman. MR. CHAPMAN.—I believe that trouble is due to the fact that the metal reinforcement which was in the form of an expanded metal lath was not protected by a sufficient thickness of dense mortar on its surface to prevent corrosion. It was a steel frame building covered with metal lath and then plastered on both sides of the lath, inside and out. I do not think the stucco was thick enough; the metal was not imbedded deep enough to protect it against moisture from the outside and the corrosion of the metal lath with its consequent swelling has ruptured the mortar through its center. The metal lath was not galvanized.

Mr. McMaster. MR. H. B. MCMASTER.—Might it not be of some interest to learn something of the elements of that plaster. If we had an

analysis of the plaster it might indicate in a way the cause. I think Mr. Chapman's explanation in a way covers the cause, but the plaster might show it still further. Mr. McMaster.

MR. CHAPMAN.—We have no record of the exact proportions or of the nature of the constituents of the plaster or mortar used. Mr. Chapman.

MR. E. D. BOYER.—In the plaster at Long Beach porous aggregates were used. The cement mortar was very porous and the water penetrated through the pores of the concrete to the metal lath. Mr. Boyer.

MR. AURYANSEN.—How much mortar would be sufficient to properly protect metal lath? Mr. Auryansen.

MR. CHAPMAN.—We are now trying to gather data to answer that question. We have exposed to the weather metal, plain iron, imbedded in mortar blocks, varying from $\frac{1}{2}$ in. covering over the metal up to 2 in., by quarters of an inch. The $\frac{1}{2}$ in. cracked some time ago. The $\frac{3}{4}$ in. has not yet cracked. They have been out now something over 2 years. It will probably take a long time to determine the period required to crack the block in which the iron is imbedded 2 in. deep. As to the amount that is sufficient to give proper protection, I would rather see 1 in. at least, and preferably $1\frac{1}{2}$ in. over any expanded metal lath, ungalvanized, that has an exposure on the south shore of Long Island. Even the latter should be a good, dense, impermeable mortar. Mr. Chapman.

MR. AURYANSEN.—Should there not be in addition to the concrete covering of $1\frac{1}{2}$ or 2 in., some paint coating, paraffine or something of that kind? Is that really necessary, or would the 2-in. covering be enough? We have a good many reinforced concrete structures building and in view. Mr. Auryansen

MR. CHAPMAN.—I do not think that anything is needed in addition to the $1\frac{1}{2}$-in. covering of a good, dense mixture, made of a good quality of sand and rich enough to be watertight. That is, not leaner than 1 : 2 ought to give ample protection to a galvanized metal lath. Mr. Chapman.

PRESIDENT HUMPHREY.—The Chair would suggest that even 2 in. is not necessary. If the concrete is sufficiently dense 1 in. is sufficient. The President.

MR. CHAPMAN.—My answer to that would be that after 2 years the $\frac{1}{2}$ in. has already cracked in our test, and 2 years is not Mr. Chapman.

Mr. Chapman. very long. I do not know how long it will be before the 1 in. goes and how much longer the $1\frac{1}{4}$ in. will stand.

Mr. Wig. MR. WIG.—I would like to ask Mr. Chapman if he does not think the form of his test piece has a good deal to do with the results, the extent of the corrosion probably being progressive from the exposed surface; or is the metal entirely covered on all sides?

Mr. Chapman. MR. CHAPMAN.—The metal is entirely covered, with a uniform thickness on all sides.

Mr. Wig. MR. WIG.—The results are so contrary to the many results obtained in connection with our sea water investigations. We have some test pieces 2 years old in which the metal placed in some cases within $\frac{1}{4}$ in. of the surface, upon removal appears as clean as when embedded. I think the density of the mortar and the way it is applied have a good deal more to do with its protecting quality than the thickness of the layer. An impermeable layer can be secured not over $\frac{1}{4}$ or $\frac{1}{2}$ in. thick, and I think it is just as good as 2 in. if it can be properly applied.

Mr. McCollum. MR. MCCOLLUM.—In regard to Mr. Chapman's question as to what might happen if these test pieces had never been broken open for examination, I think it is very significant that the ones broken after 4 months in the circuit showed the iron to be in a perfect state of preservation. Since there is no appreciable deterioration going on, I confidently expect that those test pieces will go a good many years without showing cracks, and they are to be allowed to run some years in order to test that matter out.

As to the voltage, as you say, it depends more on the form of the test piece than on the actual voltage used. Five volts impressed on a small test piece would do more harm than 25 on a much larger one. In other words it is the potential gradient that is important. I should like to ask Mr. Chapman whether or not anything was added to the water in which his test pieces were immersed.

Mr. Chapman. MR. CHAPMAN.—Nothing was added to the water. The water used was Croton water, from the hydrant. The blocks were placed in about an inch of water, in a pan about 2 in. deep, and the contacts were made with the metal pan and with the electrode in the specimen. The electrodes were imbedded about

5 in. deep and were 2 in. wide, which made an exposed area of 20 sq. in. on the two sides. Mr. Chapman.

Mr. McCollum.—There is just one point I would like to bring out, and that is that there may be some action going on there affecting the test piece, independent of the action of the electric current. In our laboratory there are a number of 6-in. concrete cylinders that have been standing in water about 1 in. deep, and there is an opportunity for water to work up through the concrete and evaporate. Some of these test pieces contain a small quantity of salt, while others do not, and those containing salt are all splitting off on the outer surface. Other test pieces made at the same time, which contain calcium chloride, showed a little of this effect but much less than when sodium chloride was present. Normal test pieces did not show it at all. We have found in our work that a very minute quantity of some impurities, as one or two hundredths of one per cent of chloride, for instance, will suffice to destroy the passive state of the iron and permit rapid corrosion. Mr. McCollum.

REPORT OF COMMITTEE ON REINFORCED CONCRETE AND BUILDING LAWS.

TESTS OF REINFORCED CONCRETE COLUMNS.

The Association has undertaken through its Committee on Reinforced Concrete and Building Laws, an investigation of reinforced concrete columns of large size. In the execution of this program the Association has been favored with the co-operation of the United States Bureau of Standards in the use of the ten million pound compression testing machine at Pittsburgh, Pa., and of the Engineering Experiment Station of the University of Illinois in furnishing observers in conducting the tests. The tests were made possible through the active co-operation of those who donated the funds and materials for making and molding the columns.

The columns were cast in New York, N. Y., on November 19, 22 and 26, 1913, and will be tested beginning February 15, 1914, at which time the concrete will be approximately 90 days old. The following is a schedule of the columns included in the series.

Hooped Columns.

Group I. Columns 1, 1a and 1b. 1 per cent spiral. No vertical steel.
Columns 2, 2a and 2b. 1 per cent spiral. 1 per cent vertical steel.
Columns 3, 3a and 3b. 1 per cent spiral. 2 per cent vertical steel.
Columns 4, 4a and 4b. 1 per cent spiral. 6 per cent vertical steel.

Group II. *Effect of increasing spiral.*
Columns 5 and 5a. 2 per cent spiral. 1 per cent vertical steel.

Group III. *Effect of protective shell.*
Columns 6 and 6a. 1 per cent spiral. 1 per cent vertical steel.

Group IV. *Plain Concrete.*
Columns 7 and 7a. No spiral. No vertical steel.

Vertically Reinforced Columns.

Group V.
Columns 8 and 8a. 1 per cent vertical steel. $\frac{1}{4}$ in. diameter hoops, 12 in. apart.
Columns 9 and 9a. 2 per cent vertical steel. $\frac{1}{4}$ in. diameter hoops, 12 in. apart.
Columns 10 and 10a. 4 per cent vertical steel. Double $\frac{1}{4}$ in. diameter hoops 12 in. apart.

The columns are 12 ft. in length and 20 in. in diameter. The diameter of shaft of columns is enlarged to 28 in. at both ends. Each end of all vertical rods was milled and fitted with an individual cast iron base plate, milled on the bottom and care was taken to keep these bases in the same plane.

The vertical steel and the hoops are mild steel fulfilling the requirements of Standard No. 7 of the Association. The spirals for the hoop columns are cold drawn wire.

The cement met the requirements of Standard Specifications for Portland Cement (Standard No. 1 of the Association).

Long Island sand and silicious gravel passing a $1\frac{1}{4}$ in. screen was used.

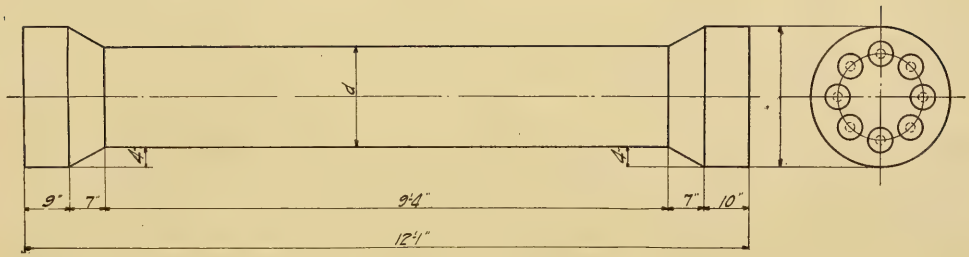
Two test cylinders, 8 in. diameter and 16 in. long, were taken from the concrete used in each column and will be tested for compressive strength and modulus of elasticity at the same age as the columns.

In order to provide a suitable base for placing the molds in which the columns were cast a special reinforced concrete slab was built to hold all the molds. The columns will be carefully placed in two flat cars and shipped to Pittsburgh where they will be stored in the testing laboratory until the tests are made.

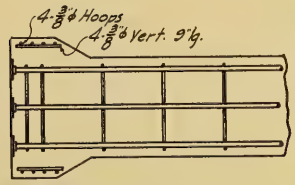
Subsequent tests will be made as found desirable after a study of the results of the tests of these columns, the purpose being to secure practical data on columns as actually used in building construction, which will serve as a basis for the recommendations of the Committee on Reinforced Concrete.

The concrete was mixed in the proportion of one part Portland cement, one and one-half parts sand and three parts gravel. The general design of the columns and details of reinforcement are shown in Fig. 1 and sections of the various types of reinforcement used, in Fig. 2.

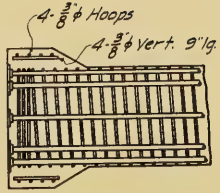
This is the largest series of columns of this size that has been tested and it is expected that the results will be of great practical value in the formulation of building regulations.



d	Columns.
1'-8"	8, 8a, 9, 9a, 10, 10a
1'-8¾"	1, 1a, 1b, 2, 2a, 2b, 3, 3a, 3b, 4, 4a, 4b, 5, 5a, 7, 7a
2'-0"	6, 6a



Section showing steel at each end for Hooped Reinforcement for Cols. 8, 8a, 9, 9a, 10, 10a.



Section showing steel at each end for Spiral Reinforcement.

FIG. 1.—GENERAL DIMENSIONS AND DETAILS OF REINFORCED CONCRETE COLUMNS.

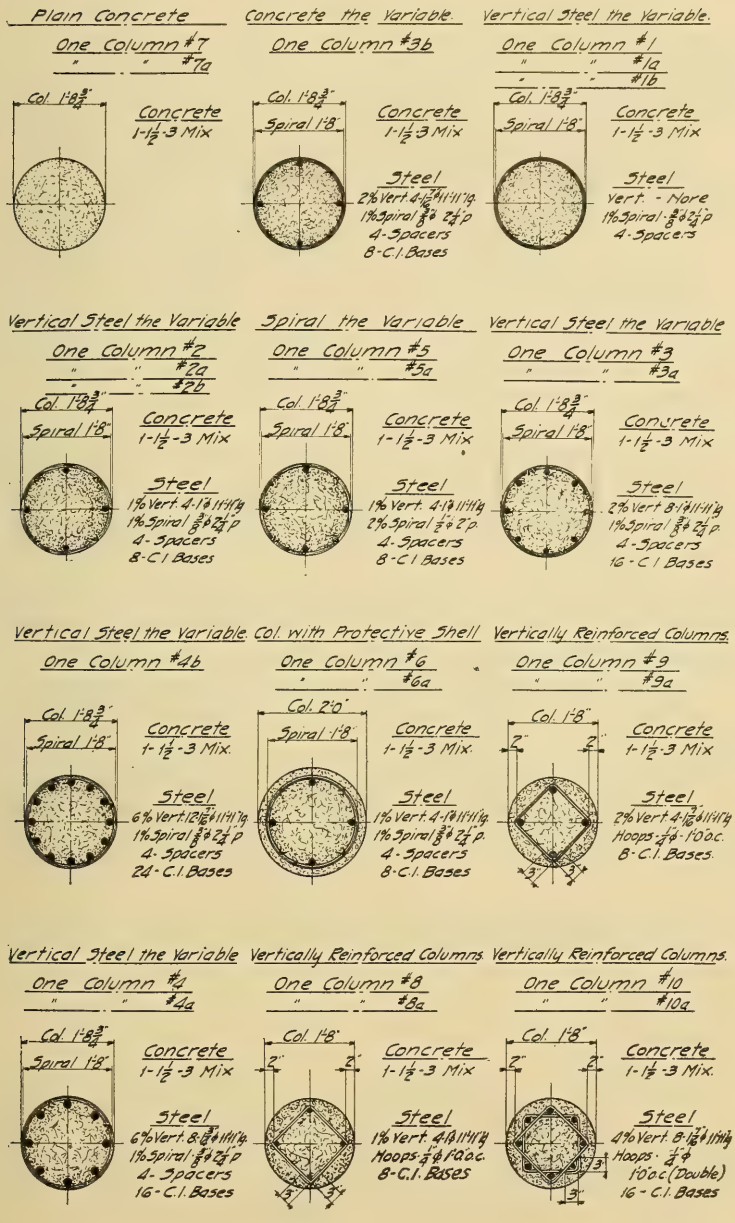


FIG. 2.—SECTIONS AND DETAILS OF REINFORCED CONCRETE COLUMNS.

PRINCIPLES OF DESIGN AND RESULTS OF TESTS ON GIRDERLESS FLOOR CONSTRUCTION OF REINFORCED CONCRETE.

By T. L. Condron.*

Prior to 1907, all of the reinforced concrete flat slab, girderless floor constructions described or built, were reinforced with four or more belts of bars or mesh, placed in four or more directions and layers, that is, in both rectangular and diagonal directions in each panel. Moreover, such constructions also included special arrangements of column head reinforcement, consisting of circular hoops or square frames combined with radial bent bars, extending out into the floor slab from the columns, through the flaring column heads. No other method of reinforcing such floors had been published either in foreign or American publications, or filed in the U. S. Patent Office as conceptions, so far as the author has been able to learn.

At the "Cement Show" in Chicago, in December, 1907, there was exhibited a model of a flat slab construction, which was publicly tested in comparison with a similar model of a girder and slab construction. These two models, of one-eighth full size, were made of concrete, reinforced with wires. One model represented the girder and slab construction of a building, the other was the first of a flat slab construction, the original two-way and two-layer reinforcement of a girderless floor, without any special column head reinforcement. The models were designed for a live load of 150 lb. per sq. ft., and the "flat slab" was tested to 900 lb. per sq. ft., under which excessive load cracks developed, but collapse did not occur. In fact, this maximum load was repeatedly applied, and the deflections carefully measured. The cracks indicated the regions of maximum tension and confirmed the analysis of stresses. In 1908, the first application was made of this construction in a building erected at Hammond, Indiana, designed for a live loading of 400 lb. per sq. ft.

*President, Condron Company, Chicago, Ill.

The method of reinforcing these floors is patented* and is shown in Figs. 1, 2 and 3. As will be seen, it consists of rectangular belts of bars in *two-directions only*. Fig. 4 shows the finished

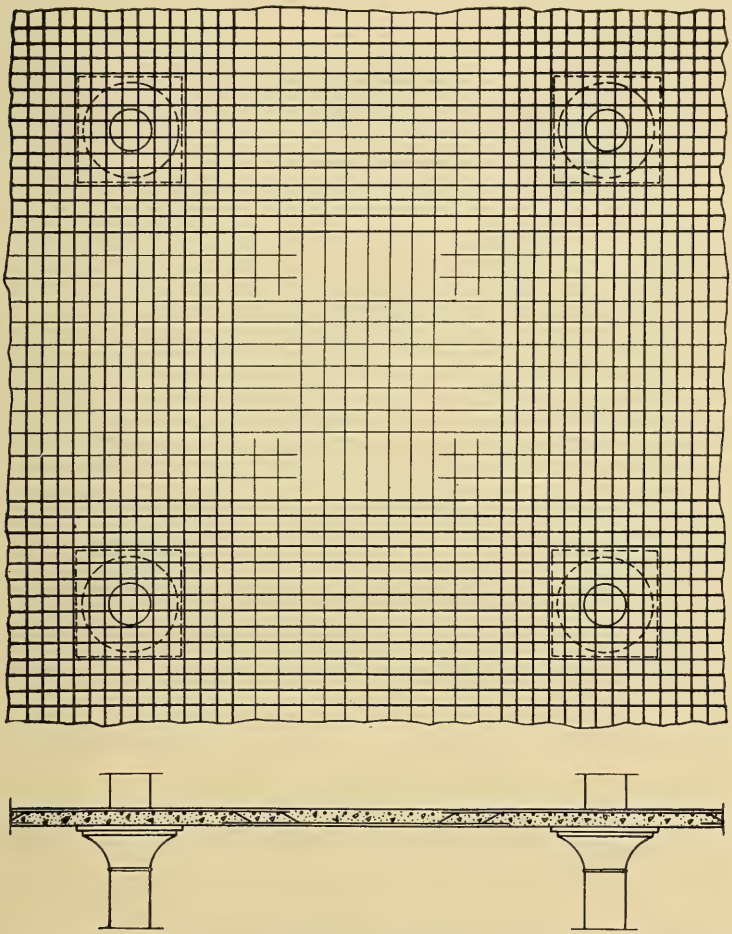


FIG. 1.—TYPICAL PLAN AND SECTION SHOWING THE ARRANGEMENT OF REINFORCEMENT.

appearance of column and slab. The main belts of reinforcement pass over the column heads and surround an enclosed space at the center of the panel which is reinforced with secondary belts

*U. S. Letters Patent 1,005,756.

of bars parallel to the main belts and extending across the latter. A feature of this construction that was entirely novel, was the transverse reinforcement of the main belts of reinforcement, so

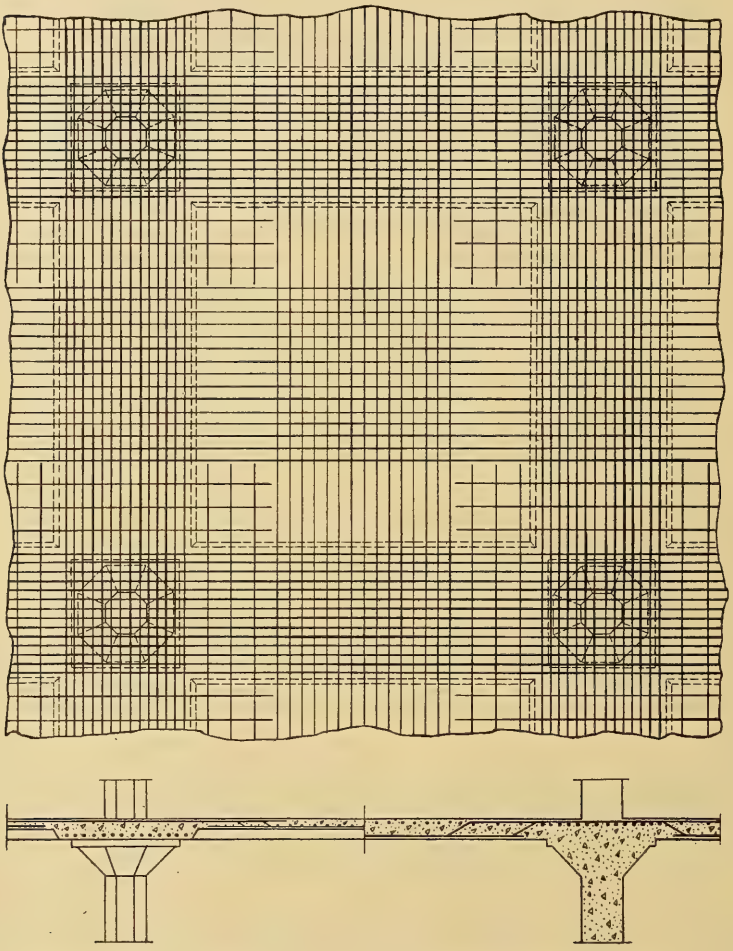


FIG. 2.—TYPICAL PLAN AND SECTION SHOWING THE ARRANGEMENT OF REINFORCEMENT.

that these belts become wide, shallow girders reinforced longitudinally to resist longitudinal tension and transversely to resist the transverse tension, due to being loaded along the edges and

supported only on part of their widths. This seems simple when it is pointed out, but until this design had been brought out and described in print, the importance and necessity of this transverse reinforcement, apparently was not realized by others.

A highly technical analysis of the stresses in a flat plate of reinforced concrete supported by a number of flaring column heads will not be attempted, but some of the fundamental considerations will be pointed out, and rational ways of determining the tension, compression and shearing stresses discussed. Fig. 5 is a diagram of a portion of such a floor supported upon six columns,

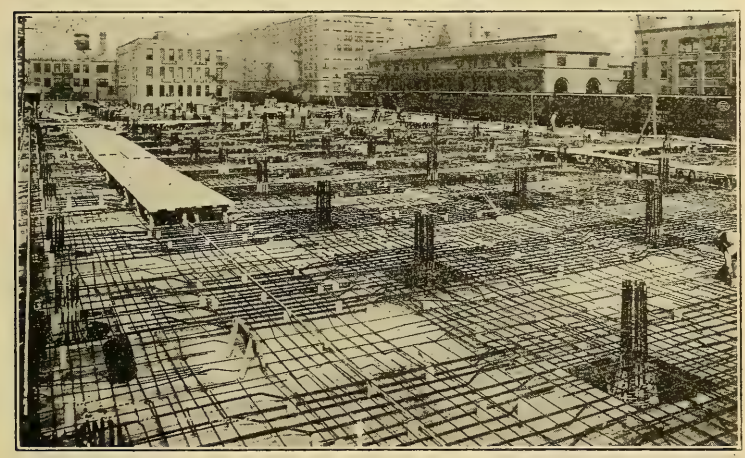


FIG. 3.—REINFORCEMENT FOR FLAT SLAB CONSTRUCTION.

and Fig. 6 is a longitudinal section across the same, while Fig. 7 is a diagram of certain moment curves. If a longitudinal element of the floor, in line with the column centers, be considered as uniformly loaded and the width of the column heads be reduced to knife edges, then the bending moments along this element of the floor will be represented by the dotted curve in Fig. 7, and the negative moment over each support will be equal to $WL/12$, where L is the distance between knife-edge supports. At the center of the span the positive moment will be $WL/24$, or one-half the moment over the support. If the supports are wide column heads instead of knife edges, the moment curve will be

changed, because the clear span will be shortened and this element of the floor will become a beam with fixed ends and a span equal the clear distance between column caps. The moment at the edge of the column cap will be equal to $WS/12$ where S equals the clear span, and the moment at the center of the span will be $WS/24$, or one-half the moment at the edge of the cap. If one panel only is loaded, the moment will presumably be greater at the center of the span, than if several

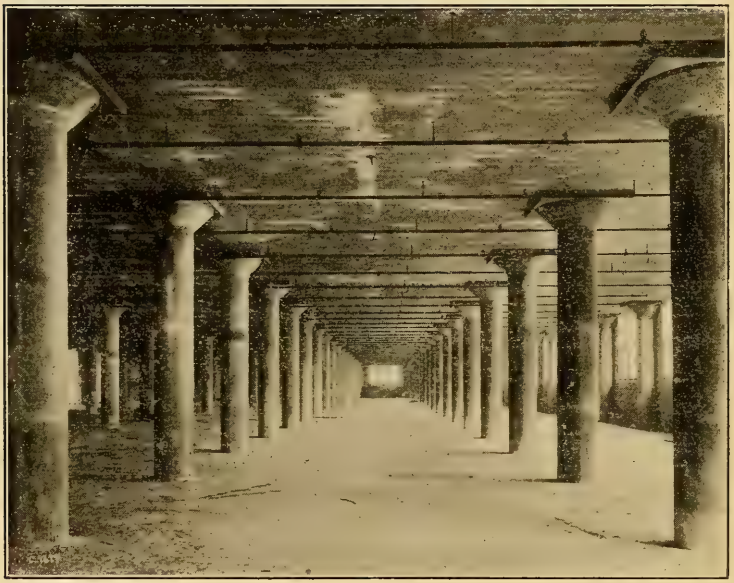
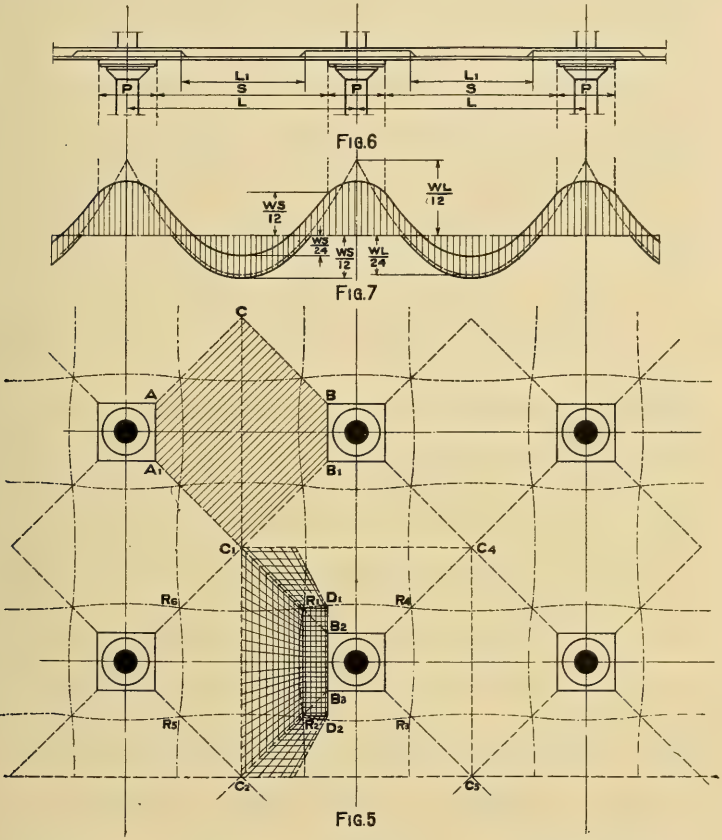


FIG. 4.—TYPICAL INTERIOR FLAT SLAB CONSTRUCTION. NOTE THE PERFECT LIGHTING OF THIS ROOM, WHICH IS 364 FT. LONG BY 127 FT. WIDE.

adjacent panels are loaded, and to meet this condition of loading assume a moment at the center of the span equal to the moment at the edge of the column cap.

If the columns are placed in the usual manner, as indicated in Fig. 5, each column will support a section of floor, C1, C2, C3, C4; and if the column heads are rectangular, there will be supported on each edge of the column head, an area of the floor equal to A, C, C1, A1.

Two simple ways are suggested for analyzing the stresses in such a slab supported upon rectangular column heads and with rectangular reinforcement, either of which follows the ordinary methods of beam analysis, does not involve abstruse reasoning



FIGS. 5, 6 AND 7.—DIAGRAM OF A PORTION OF A FLAT SLAB FLOOR, SECTION THROUGH THE SAME AND THE MOMENT CURVES, DESCRIBED IN THE TEXT.

and may be relied upon to give safe and sane results, nearly enough correct for all practical considerations.

The first method is to consider the loaded area A C B B1 C1 A1, carried by the broad girder AA1–B1B having the clear span

AB and having its ends fixed. Considering this girder uniformly loaded, the moment at the edge of the column plate will be $WS/12$. Since W equals the load on one-half the panel (exclusive of the area of the column plate) it is equal to $W/2\ (L^2-P^2)$. Therefore, the moment at the edge of the cap is equal to $W/24\ (L^2-P^2)\ S$, or $W/24\ (L^2-P^2)\ (L-P)$. If $P=L/4$, the moment becomes equal to $WL^3/34$.

The other method of analysis is to consider that the slab supported by the several column caps is in negative bending over the columns, within the lines of inflection R1 R2−R2 R3−R3 R4 and R4 R1, and in positive bending between these lines and corresponding lines about the adjacent heads. Then assume that these lines of inflection are located at a distance from the column centers, which is a mean between $0.2L$ and $0.2S+P/2$. Since $S=L-P$, this distance is equal, $0.2L+0.15P$. If $P=L/4$ then the distance from column center to the lines of inflection becomes $0.2375L$.

By this method, the moment at the edge of the cap, when the width of the column cap $=L/4$, becomes $WL^3/33$, or practically the same as by the first method. The calculations by this method are somewhat complicated for the general case, but are simple for specific dimensions.

After the moment at the edge of the cap is determined and if the same is arbitrarily assumed at the mid-span for each belt of bars, the slab will be over-reinforced at midspan between columns for all panels loaded, which introduces an extra degree of safety. The reinforcement of the region enclosed by the four main belts is readily determined, this portion being a slab reinforced in two directions. The moment between the lines of inflection will be $Wl^2/8$ or $Wl^2/16$ for each direction where l is the distance between lines of inflection.

The importance of transversely reinforcing the top of the slab, across the main belts of bars, has already been mentioned and too much emphasis cannot be laid upon this feature.

The proper width of the belts of reinforcement is a matter of judgment based upon experience and deflections under tests, but these belts will be wider than the caps, but not wider than the distance between the lines of inflection. The thickness of the slab is determined by limiting the shear at the plane of contra-

flecture about the column to less than 40 lb. per sq. in. on the effective section.

The area of reinforcement required is determined under the allowed unit stress, for each of the two layers of bars over the column plate, as the difference of the resisting arms for the two layers should, in our opinion, be taken account of. Efficient supports for the upper reinforcement are provided which we believe in maintaining in a truly horizontal position, where it is needed to resist tension. Therefore, the designs call for the accurate bending and placing of the bars. Under the Chicago building ordinances, the stresses in steel bars are limited to a maximum of 18,000 lb. per sq. in., for high carbon and for cold twisted bars, and the compression in 1 : 2 : 4 concrete is limited to 700 lb., but for compression on the under side of the slab at the region of maximum reverse moment, this limit is increased to 750 lb. Under the rules laid down for this construction by the Commission consisting of A. N. Talbot, W. K. Hatt and L. E. Ritter, this compression is measured on a width equal to the width of the cap, plus ten times the thickness of slab, but not more than two-thirds the distance between column centers. As the whole under side of the slab is in compression transversely along the rectangular lines between columns and the upper surface is correspondingly in tension, it is doubtful if the allowable tension or compression is ever reached within any considerable portion of these regions, for the stresses must gradually extend throughout the slab, and not reach sharply defined maximums as in the case of separate beams and girders.

What has been said here applies particularly to the type of flat slab covered by this paper and will not equally apply to four-way reinforcement, but the stresses around the column heads will be much the same for both systems, and as much reinforcement should be used in one as in the other, except that it is believed material advantage is gained by limiting the reinforcement to two layers in this region and in using square column plates over which the rectangular reinforcement passes.

In conclusion, the records of a number of carefully conducted floor tests of this construction are submitted, in which no attempt has been made to measure anything but the deflections of the slabs at various points. These results are divided into two groups.

First, the tests of strictly flat slabs, and second, tests on a special type of paneled slab. In the latter, the slab is reduced in thickness in its central portion, permitting it to be thicker between columns for the same quantity of concrete. This type naturally deflects less than the strictly flat slab, but the form work costs more, and for places where pipes and shafting are to be hung the flat ceiling is much better.

All but one of the tests given in Tables I and II were made on floors of buildings in Chicago, under the direction of the Chicago Building Department. The Chicago building ordinance, adopted in December, 1910, states that:

> Floor joists and beams shall be designed for the full dead and live loads. Floor girders shall be designed for the full dead and not less than 85 per cent of the live load.

Regarding tests, the ordinance says:

> Tests shall be made by the owner upon the demand of the Commissioner of Buildings on all forms of construction involving spans of over 8 ft. Such tests shall be made to the approval of the Commissioner of Buildings and must show that the construction will sustain a load equal to twice the sum of the live and dead loads, for which it was designed, without any indication of failure. Each test load shall remain in place at least 24 hours.

Under the provisions for reinforced concrete, there is a special test requirement which is similar to the one above, but in addition, contains the following:

> Each test load shall cover two or more panels and shall remain in place at least 24 hours. The deflection under the full test load at the expiration of 24 hours shall not exceed 1/800th of the span. These tests shall be considered as tests of workmanship only.

From the paragraphs quoted, it will be noted that the floor girders may be designed for 85 per cent of the live load, while the floor between girders shall be designed for the full live load. Under a test load covering two panels the girder between the panels would be stressed twice as much as would be the girders under a test load covering one panel.

Prior to eighteen months ago, the department did not require tests covering more than one panel, but now it is frequently required that tests shall cover two panels. Up to the time of the

adoption of the ordinance, no tests had been made in Chicago covering more than one panel, and, therefore, the deflection requirement was based on the results of single panel tests of beam and girder construction.

The last two tests in Table II were both on the same floor. First, one panel only was loaded with 500 lb. per sq. ft. This load was allowed to remain for 24 hours, causing a deflection of 0.33 in., then the adjacent panel was loaded. After the load had been on the first panel 48 hours and on the other 24 hours, the deflection of the first panel had increased from 0.33 to 0.44 in. and the second panel had deflected 0.40 in., showing that the loading of two panels caused a greater deflection in both than would result from a single panel test. All of the floors in this table, except the first, were designed under the rules of the ordinance, for 85 per cent of the live load over the entire floor and the full live load over any one panel, therefore one of the belts of reinforcement in each of these two panel tests is stressed up to about 2½ times the load for which it is designed.

It is certainly not right to subject buildings to such severe loading tests. A test load equal to twice the live load (where the live load is greater than the dead load) seems to me to be the maximum test load that should be called for. With regard to the proper amount of deflection which should be considered satisfactory, under test load, this can only be arrived at by careful study of the many tests that have been made on various types of construction. The permissible deflection, under test load, should be less than the deflection that produces visible cracks in the finished concrete surface of the floor or ceiling. Reinforced concrete structures should not be subjected to loadings that will produce visible cracks, and certainly structures should be so designed that working loads will not produce cracks.

In all of the tests reported here, careful examinations have failed to disclose even fine cracks in the floor surfaces, and in only one instance have there been seen even hair cracks in the ceiling. Under the 625 lb. per sq. ft. test load (Fig. 8), where the ceiling had been painted with cold water paint, hair cracks were visible at the center of the panel, but disappeared from sight after the load was removed. This floor was designed for 250 lb. live load.

In addition to measuring the deflections at the centers of

panels, the deflections at the middle points between columns have been observed on many of these tests. These deflections are generally from one-fifth to one-fourth as much as the deflections at the centers of the panels for square panels, except that the deflection between columns at the center of a double panel test is about 90 per cent of the deflection at the center of the panel. Where the panel is not square, the deflection at the center of the line between columns on the long span may be as much as one-half of the deflection at the center of the panel.

From results obtained, a limit of 1/800th of diagonal span for a single panel test of double the live load would be entirely reasonable, but is apparently too severe a limitation where the test load is made to cover two panels and is equal to twice the live, plus the dead load. For such a test, the permissible deflection should be at least 1/600th of the diagonal span.

In presenting these principles of design and the results of tests, it is with the desire to contribute something useful to the general study of reinforced concrete floor design. It is hoped that the papers and discussions on this subject will lead to a clearer understanding of reinforced concrete beamless floor construction, which has such great strength and so many advantages over girder and beam construction.

MEASUREMENT OF ACTUAL STRESSES IN A CANTILEVER FLAT SLAB REINFORCED CONCRETE FLOOR HAVING RECTANGULAR PANELS.

By Arthur R. Lord.*

1. Building Tested.

The Chicago Warehouse of the Larkin Company, in which the test described in this paper was made, is an eight-story and basement building of reinforced concrete construction throughout. The building is 200 by 125 ft. in plan, divided into panels 20 ft. by 24 ft. 2 in. in dimension. The type of reinforced concrete construction used is that known as "Cantilever Flat Slab," and the designers were the Concrete Steel Products Company of Chicago. The Leonard Construction Company of Chicago were the general contractors. The floor tested was the second above the ground as indicated in Fig. 1. The building was designed in accordance with the building ordinances of the City of Chicago which govern cantilever flat slab construction. The slab thickness, panel dimensions, size and thickness of depressed head, size of column capital and shaft, the section of the adjacent concrete wall, and other physical data of the floor tested are given in Figs. 2 and 3. The amount and distribution of the reinforcement can be obtained from the Concrete Steel Products Company, who designed the building and who caused this test to be made as a part of their extensive testing program.

2. Objects of the Test.

The writer has tested buildings in the past in which the panels were other than square and data from these tests were used in the design of this building. But in none of these buildings did the length of the panel exceed the breadth by so large a percentage as was the case in the Larkin Warehouse and in no case was the panel quite so large. Only one of the writer's previous

* Structural and Testing Engineer, Chicago, Ill.

tests has included wall panels. The test was planned accordingly with two chief objects in view: *first*, to secure data on the distribution of stress in the various directions in a flat slab floor having decidedly rectangular panels, and *second*, to ascertain the relative magnitude of the stresses in wall and interior panels. Various other points of more purely scientific interest such as the determination of the point of inflection, the effect of the depressed head construction, the stress in the column capital and column shaft,

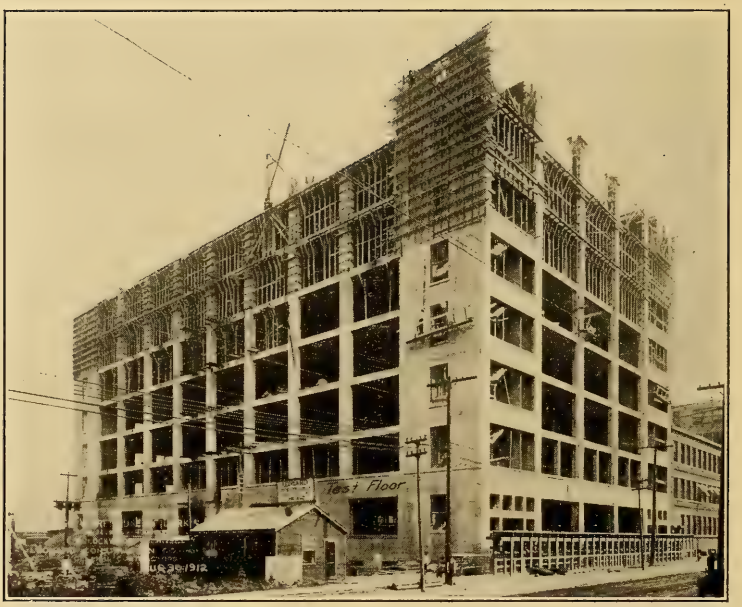


FIG. 1. THE CHICAGO WAREHOUSE OF THE LARKIN COMPANY.

the action of adjoining unloaded panels, etc., were inquired into, but the emphasis was placed on the two points mentioned above.

3. OBSERVERS.

Each observer on the test had had previous experience in work of this kind in addition to long experience in the testing laboratory at the University of Illinois. Professor W. A. Slater gave most valuable assistance throughout the test, and Messrs.

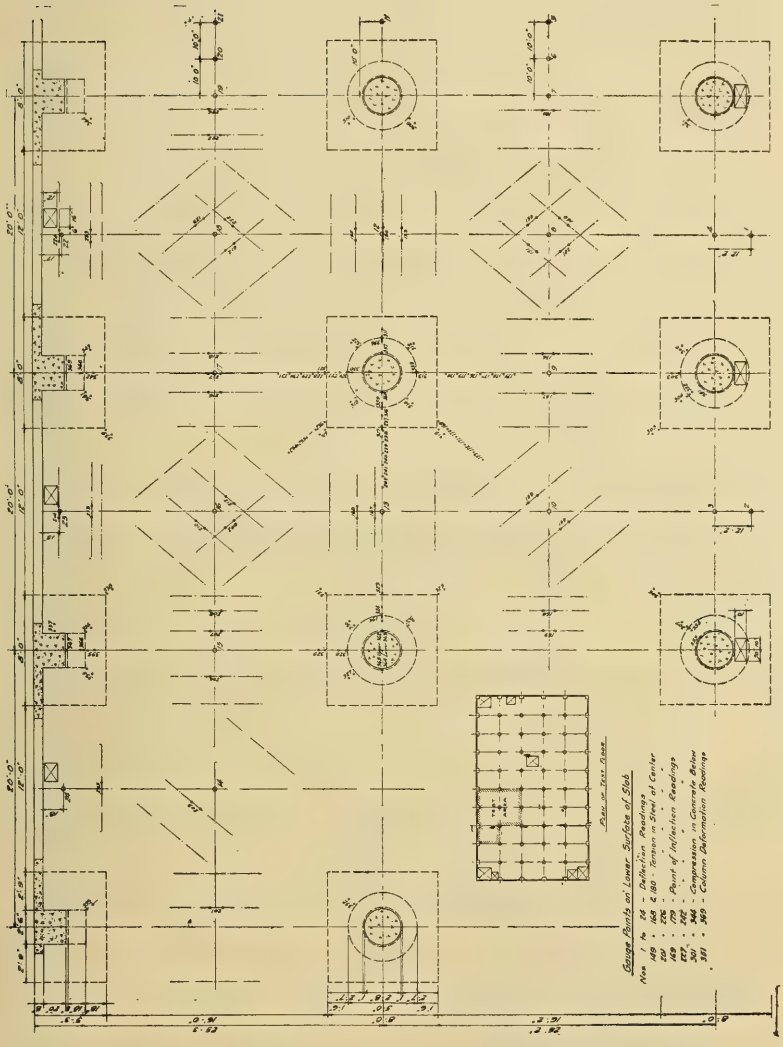

Gauge Points on Lower Surface of Slab
Nos. 1 to 124 - Deflection Readings
" 125 " 140 - Tension in Steel at Center
" 141 " 176 - Point of Inflection Readings
" 177 " 192 - Compression in Concrete Below
" 301 " 344 - Column Deformation Readings
" 351 " 360 -
Base of Test Floor

FIG. 2.—GAUGE POINTS ON LOWER SURFACE OF SLAB.

Ensign and Gonnerman, also of the University of Illinois Engineering Experiment Station, assisted Mr. Slater and the writer in observing stresses. Professor Talbot kindly reviewed the plans, offered many valuable suggestions, and was a frequent visitor during the progress of the test.

4. OBSERVATIONS AND LOADING.

Fig. 2 shows diagrammatically the position and nature of the observations taken on the under surface of the test slab. These readings were taken by observers working on a scaffold erected for the purpose. Fig. 3 gives the same information for the observations taken on the upper surface of the slab. In order to make more extensive observations on the upper surface than had hitherto been attempted large areas of the floor were covered over, and the observers worked under the test load which was transmitted to the floor by means of numerous posts, giving practically uniform loading. The attempt to determine the point of inflection from deflection readings, always unsatisfactory in the past, was abandoned in planning this test and measurements of stress over consecutive gage lengths along the direct and diagonal center lines of the panel, and on both the upper and lower surfaces were substituted instead, with very gratifying results. For the upper readings of these series special bars were laid near the top of the slab, thus avoiding the disturbance of readings due to the breaking of the concrete in tension under heavy test loads. Inserts were placed on the reinforcing rods and in the concrete at points where readings were desired at the time when the concrete was poured, and by this means the time and labor of the final preparations were very materially reduced. All the measurements of deformation were made with extensometers of the University of Illinois—Berry type, and the deflections were measured with the Illionis type of deflectometer, both of which instruments have been described before.*

The sand used for loading the test panels was very well graded and extremely dense. It was slightly damp and would stand for a considerable time at an angle of 75 to 80 deg. with the horizontal. The containing bins were tied with rods and not braced back on the adjoining panels. When the bin tie rods were loosened under maximum load no appreciable movement took

* *Proceedings*, Vol. VIII. 1912, pp. 63 and 195.—ED.

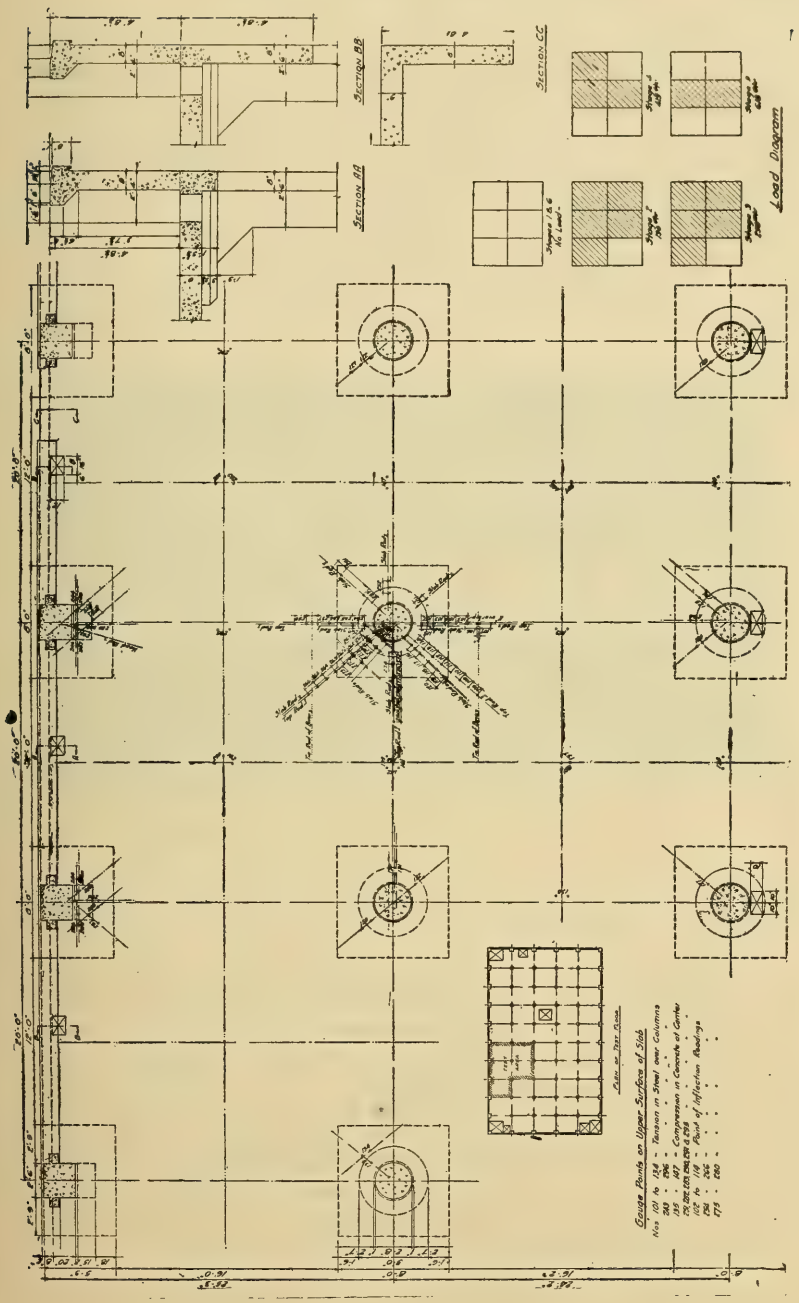

Section AB
Section CC
Section AA
Load Diagram
Gauge Points on Upper Surface of Slab.
Nos 101 to 134 = Tension in Steel over Columns
" 135 " 146 = Compression in Concrete at Centre
" 147 " 166 = Tension in Concrete at Edge of Slab
" 167 " 200 = Rod of Deflection Readings
" 201 " 250 = " " " "
" 251 " 270 = " " " "

FIG. 3.—GAUGE POINTS ON UPPER SURFACE OF SLAB. LOADING DIAGRAM.

place, which would indicate that the arch action in the load itself was very slight. Small quantities of the sand measured previous to the test gave a weight of 97 lb. per cu. ft., and the measurement of 130 cu. ft. of the actual load in place gave a weight of 101 lb. per cu. ft., as would be expected after the handling and trampling of the workmen in shifting the load. Fig. 4 is a view of the bins partially filled. In Fig. 3, in the lower right hand corner, the various stages in the loading at which observations were taken are given. The floor was designed for a live load of 225 lb. per sq. ft. and a maple floor on fill, the weight of which was considered to be

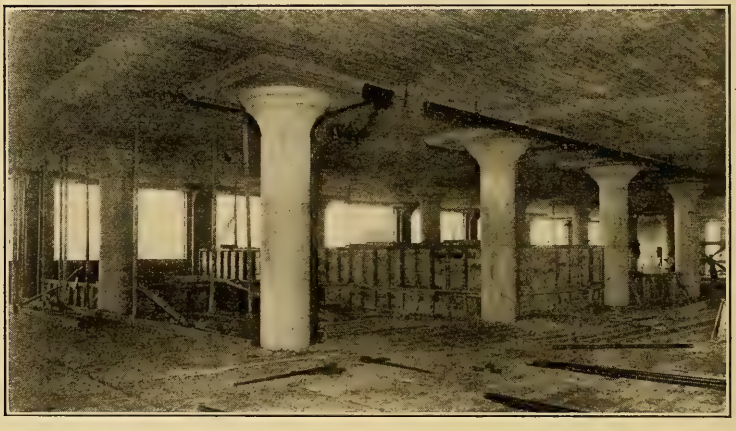


FIG. 4. VIEW OF THE BINS PARTIALLY FILLED.

25 lb. per sq. ft. This maple floor was not in place at the time of testing.

Preparations for the test occupied somewhat over a week. Two entire days were spent in getting the zero readings accurately, as these are the most difficult and important on the test. Two-hundred and fifty-five gage lengths were read throughout the test, involving more than 18,000 individual observations. At each stage in the test the notes were calculated and check readings taken whenever they seemed valuable, before the loading was resumed. For many of the important gage lines the load-deformation curves were plotted as the test progressed. All signs of stress were carefully searched out and recorded.

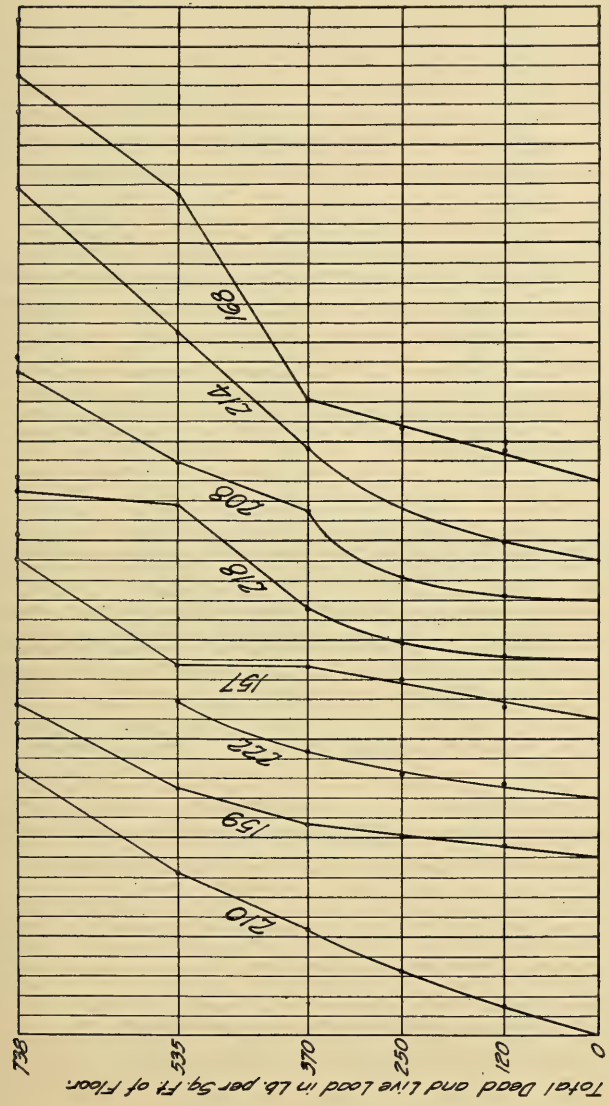


FIG. 5.—STRESS IN REINFORCEMENT AT CENTER OF SPAN.

5. QUALITY OF THE CONCRETE.

The materials used in the making of the concrete were of very good quality. The coarse aggregate was $\frac{3}{4}$-inch crushed hard limestone, the cement was a standard Portland, and the sand was clean and graded from large to small particles giving an exceptional density. The mixture was 1 : 2 : 4. At the time the test panels were being poured specimens of the concrete in the form of cubes, prisms, and beams were taken from the concrete placed in the floor. These were tested immediately after the conclusion of the test, at an age of 62 days, and gave a crushing strength in excess of 3,300 lb. per sq. in. and an initial modulus of elasticity of 4,000,000 lb. per sq. in. Other specimens taken from the seventh floor, poured at the time of the test, and tested at the same age gave the same results. The modulus as found by test has been used in the computation of the concrete stresses in the report of this test.

6. MEASURED STRESSES.

Within the limits of this paper it is not possible or desirable to give a complete record of the measured stresses. Figs. 5 to 16 are intended to furnish representative examples of the various measurements made. In Table I the results of all the measurements are summed up. In considering the load-deformation-graphs constant reference should be had to Figs. 2 and 3 keeping the location of the gage line with respect to the loaded areas of the floor in mind.

For convenience in the use of these data for design purposes the stresses have all been given for the total *dead and live* load on the floor. The dead load stress has been taken as equal to that observed under a live load equal to the weight of the slab (120 lb. per sq. ft.). In the following discussion and diagrams readings of like character have been grouped together.

(*a*) *Stress in Reinforcement at Center of Span.*—See Figs. 2 and 5. The total number of gage lines read was 37.

(*b*) *Stress in Reinforcement over Column Capital.*—See Figs. 3 and 6. The total number of gage lines read was 39. Reading 293 was taken on one of the radial head rods at a wall column. Reading 342 is included in Fig. 6 by mistake.

(*c*) *Stress in Reinforcement over Edge of Depressed Head.*—

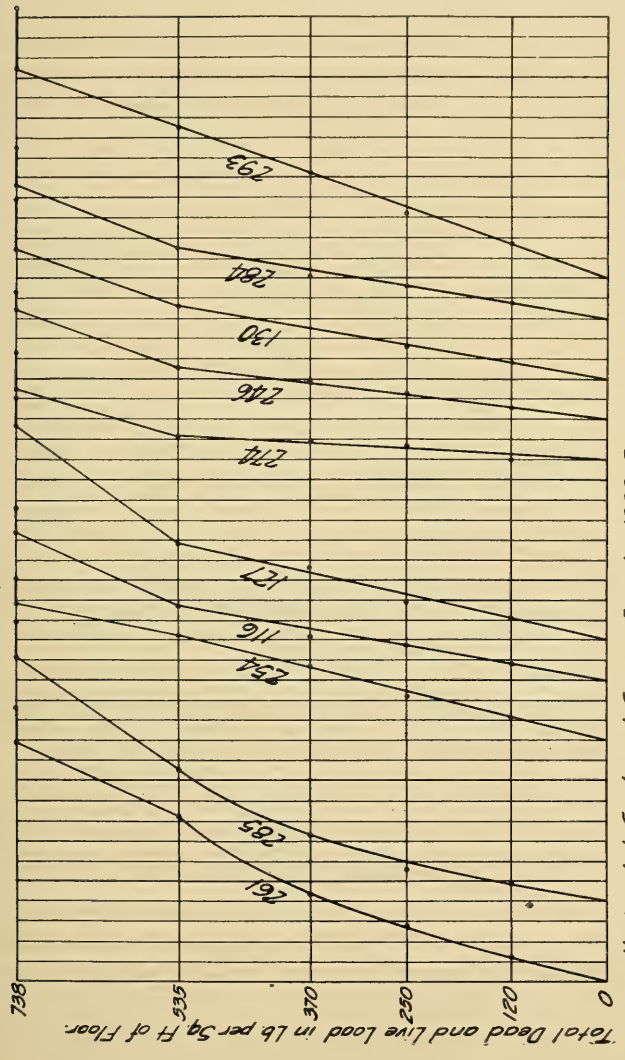


FIG. 6.—STRESS IN REINFORCEMENT OVER EDGE OF CAPITAL

See Figs. 3 and 7. The total number of gage lines read was 7. Reading 253 was taken across the column capital on the same rod as reading 281 and is introduced on this figure for comparative purposes only. The higher stress over the edge of the depressed head is due in part at least to the fact that the reinforcement at this point is farther from the upper surface of the slab and hence has a smaller moment arm.

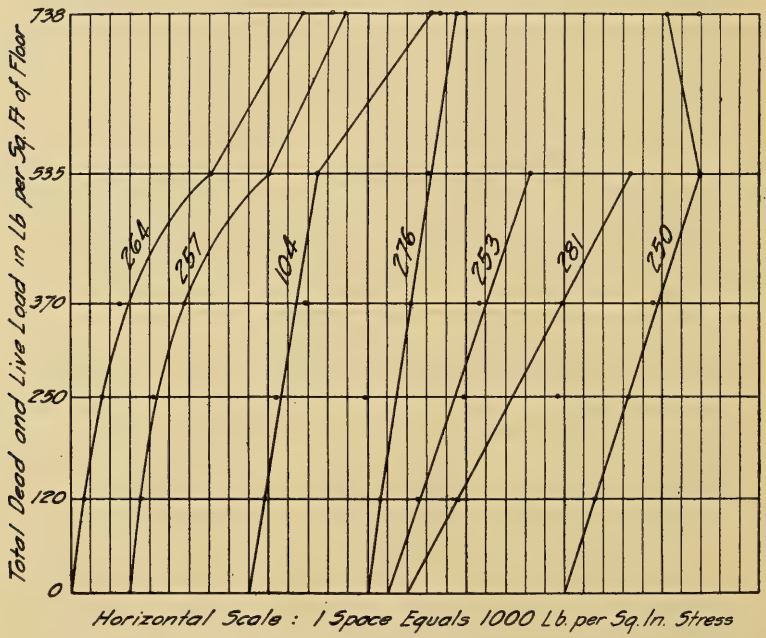


FIG. 7.—STRESS IN REINFORCEMENT OVER EDGE OF DEPRESSED HEAD.

(*d*) *Stress in Concrete at Center of Span.*—See Figs 3 and 8. The total number of gage lines read was 17. Gage lines 140 and 142 showed very slight indications of stress in tension.

(*e*) *Stress in Concrete at Edge of Capital.*—See Figs. 2 and 9. The total number of gage lines read was 30. The gage line in each case was set on the depressed head with one point close to the outer edge of the column capital. Nearly all the concrete readings on the lower surface of the slab were taken over a 6-8 in. gage length.

(*f*) *Stress in Concrete at Edge of Depressed Head.*—See Figs. 2 and 10. The total number of gage lines read was 19. The gage lines were taken on the under surface of the slab, just outside the depressed head and extended radially from the column.

(*g*) *Stress in Concrete at Top of Column Capital.*—See Figs. 2 and 11. The total number of gage lines read was 9. The upper gage point was situated about 2 in. from the under surface of the

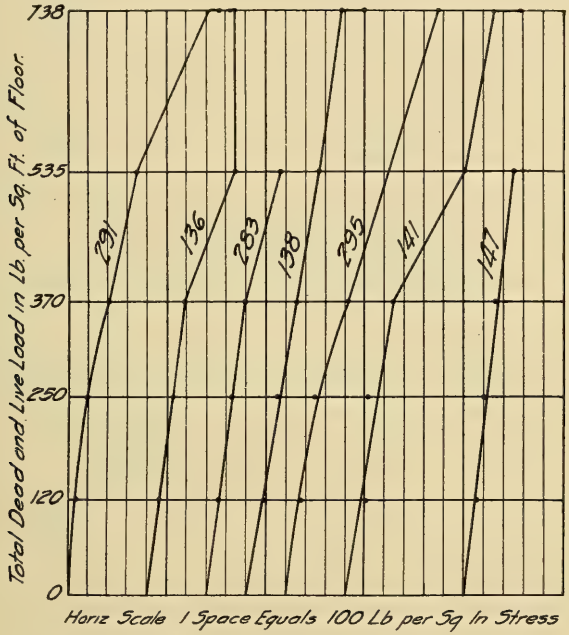


FIG. 8.—STRESS IN CONCRETE AT CENTER OF SPAN.

depressed head and the other point of the same set directly beneath it on the capital.

(*h*) *Point of Inflection Readings.*—See Figs. 2, 3, 12 and 13. Point of inflection stress readings were taken on both the upper and lower faces of the slab in five directions along the direct and diagonal center lines of the panels. The results were very satisfactory and indicate that the point of inflection may be determined from stress measurements with far greater accuracy than from

deflection measurements. Figs. 12 and 13 each show load-stress curves in one direction for four stages of the loading. As all the curves on one diagram are plotted from a common zero axis the intersection of like lines gives an indication of the position of the point of inflection in that direction, so that each diagram gives four indications of the point of inflection. The agreement is excellent.

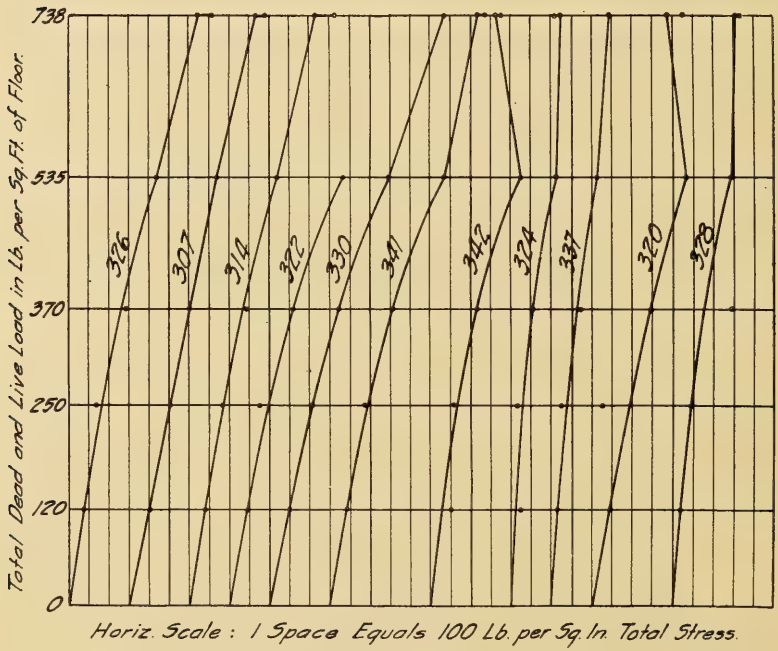


FIG. 9.—STRESS IN CONCRETE AT EDGE OF CAPITAL.

(*i*) *Deflections.*—See Figs. 2, 15 and 16. The total number of points at which deflections were measured throughout the test was 24. In Fig. 16 a large number of these measurements are plotted against load up to full load on two panels. In Fig. 15 three important readings are plotted for the entire period of the test and for one week subsequent to the removal of the load. The upper part of the diagram is plotted to a scale of time. Four sets of readings were taken under full load (twice the dead and live

design load) and three sets after the load was removed. In Fig. 15 note that the deflection decreased during the last five hours that the full load remained on the floor. A few days after the test load was removed it became necessary to place the cinder fill and floor strips for the maple flooring and these were held in place by posts and wedges driven hard against the slab above. A material increase in the deflection resulted. At the time the last

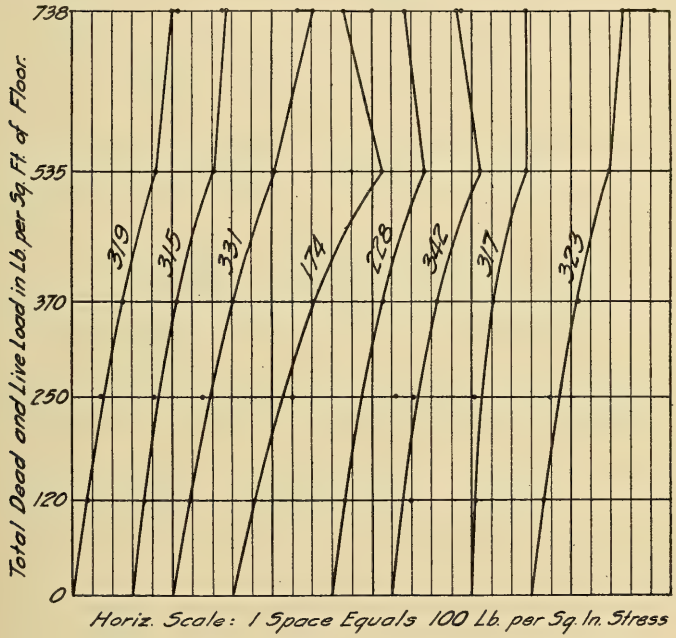


FIG. 10.—STRESS IN CONCRETE AT EDGE OF DEPRESSED HEAD.

reading was taken the posts had been removed a short time and the weight of the cinder fill and sleepers alone was on the floor.

7. COMMENTS AND CONCLUSIONS.

The writer feels that statements of opinion and personal conclusions should be excluded from a report of a scientific test. Accordingly only brief mention will be made here of a few of the observed facts not mentioned thus far. The test has afforded some very valuable design data and very interesting mathematical

relations can be deduced from the data obtained, but the present paper must leave this matter untouched.

(*a*) *Arch Action.*—In previous tests the writer has measured compressive stresses on lines directly between columns which would indicate the presence of considerable arch action, but which did not determine the amount closely. The point of inflection readings, Figs. 12 and 13, and the others not given in this paper, however, give the best arch action measurements that have so

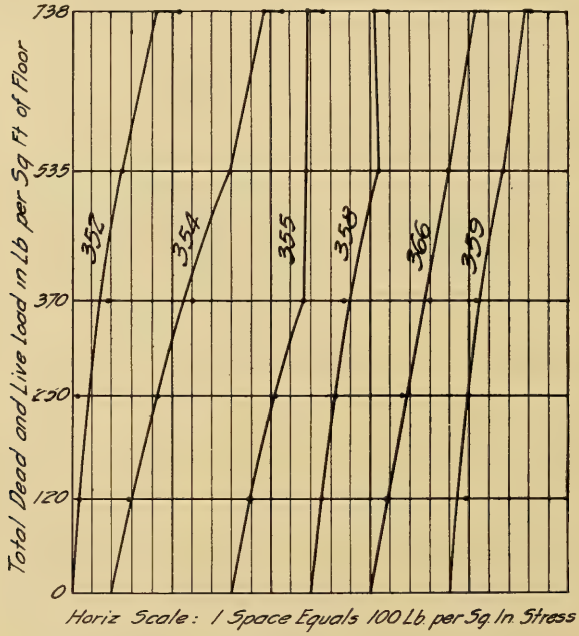


FIG. 11.—STRESS IN CONCRETE AT TOP OF COLUMN CAPITAL.

far been obtained. Fig. 13 does not give a positive indication, but Fig. 12 and two of the other series of readings show both the upper and lower surfaces of the slab to be in compression for a considerable distance on either side of the point of inflection. The average compression on the section at the point of inflection is indicated as from 150 to 250 lb. per sq. in. The other two series of readings indicate more nearly normal beam action.

(*b*) *Tension in Concrete.*—As in other tests for actual stresses

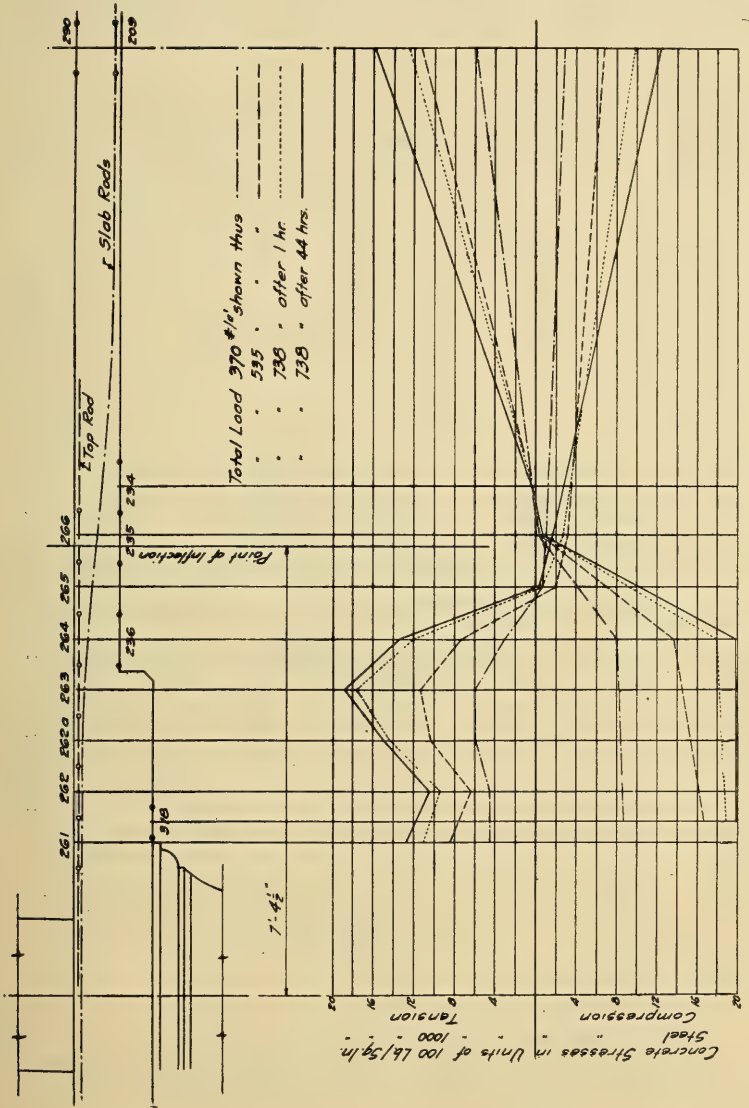

Concrete Stresses in Units of 100 lbs/sq.in.
Steel " " 1000 " " "
Compression
Tension
Total Load 370 #'s shown thus
" " 535 " " "
" " 738 " after 1 hr.
" " 738 " after 48 hrs.
Point of Inflection
T Top Rod
T Slab Rods
7'-4½"

FIG. 12.—POINT OF INFLECTION READINGS IN DIAGONAL DIRECTION ACROSS WALL PANEL.

it is evident from the measured stresses that a great deal of the load is carried by the moment of the plain concrete slab and by arch action. The compressive stresses are altogether greater

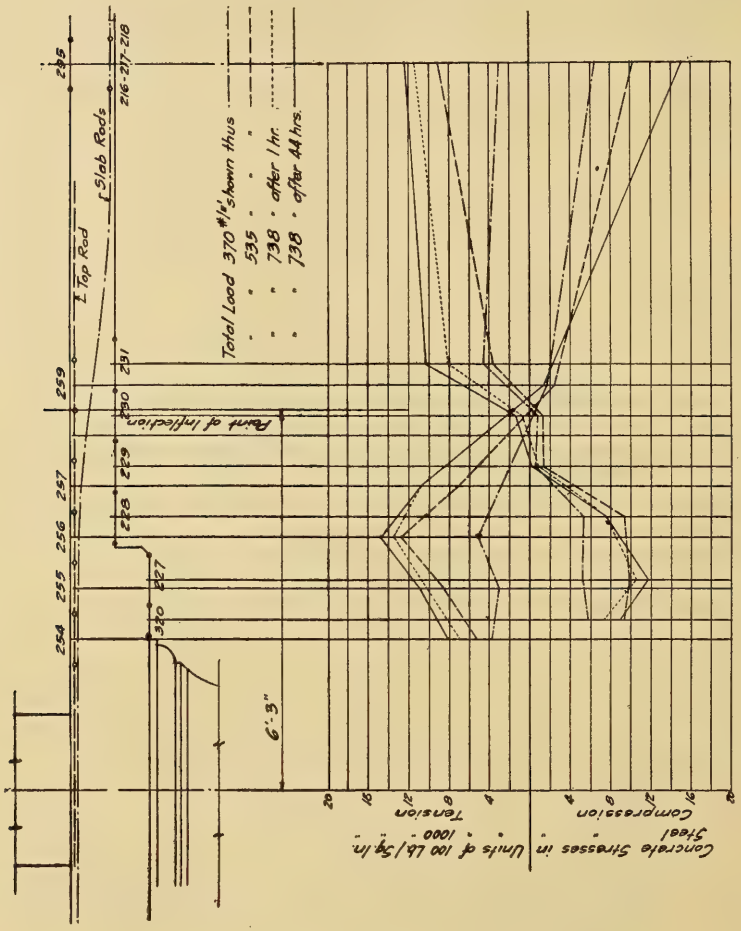


FIG. 13.—POINT OF INFLECTION READINGS IN LONG CROSS DIRECTION WALL PANEL.

than the tensile stresses in the reinforcement would lead us to expect.

(*c*) *Point of Inflection.*—In Fig. 14 the position of the point of inflection along five direct and diagonal center lines has been

plotted and the proportionate span computed. Ratios are given for the span between column centers, for the span between edges of column capitals, and for the span between edges of depressed heads.

The writer has been interested for a long time in the statement made in a book written by Mr. C. A. P. Turner, entitled "Concrete Steel Construction," and repeated often in the subse-

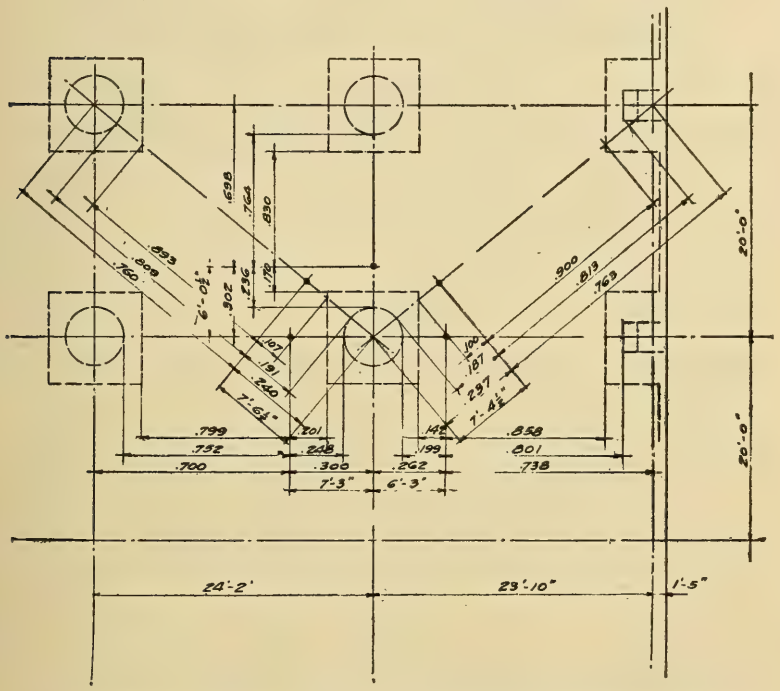


FIG. 14.—PROPORTIONATE DISTANCES TO LINE OF INFLECTION IN FIVE DIRECTIONS.

quent writings of Mr. Turner, to the effect that the section of maximum stress on the diagonal bands of flat slab construction lies at a point very close to the probable position of the line of inflection if we consider the bands as beam strips. The writer of this paper has never found any visible indications of stress at this point in any of his tests, and in the Larkin Warehouse the point of inflection readings were planned so as to measure stresses directly

over two such regions. The readings indicate that this is a section of very low stresses.

(*d*) *Distribution of Stress in Various Directions.*—The designers

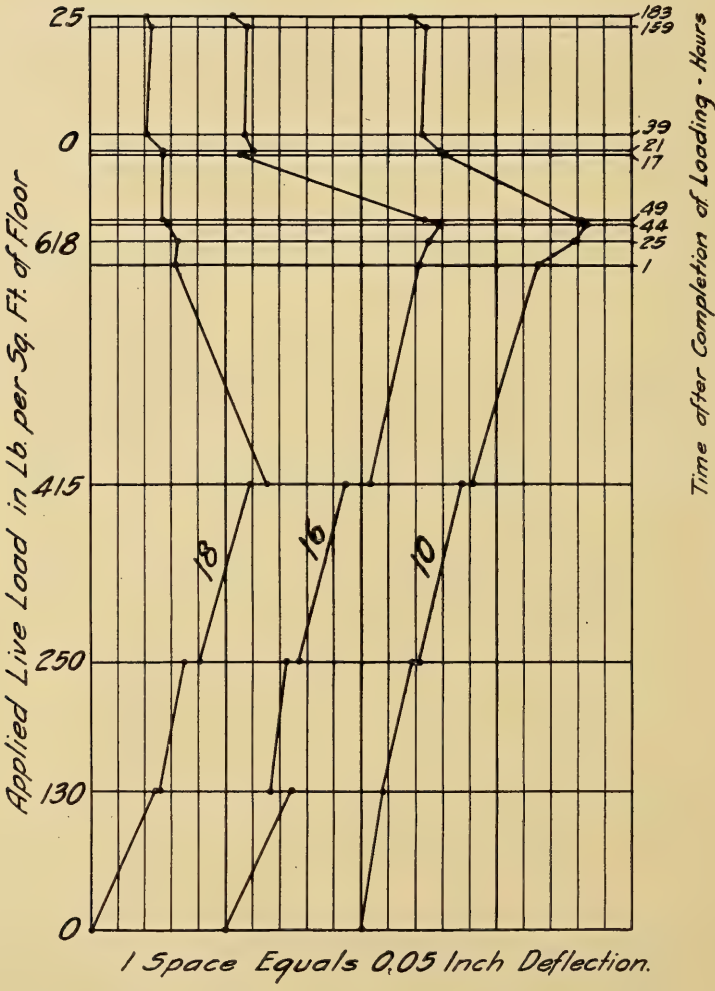


FIG. 15.—DEFLECTIONS AT PANEL CENTERS.

of the Larkin Warehouse do not care to give out the reinforcement for the floor slab tested, knowing from past experience that such data is subject to abuse on the part of irresponsible designers.

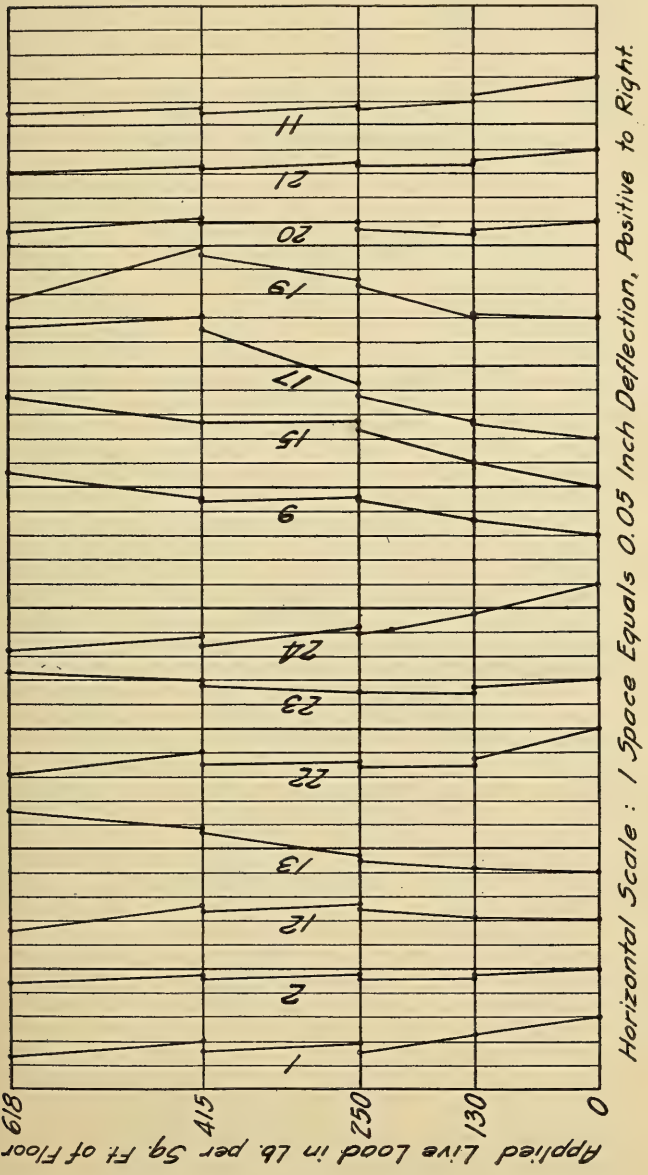


FIG. 16.—DEFLECTION READINGS.

We may arrive at a clear conception of the relative moments in the various directions, however, by considering what the stress per square inch in the reinforcement would be if the observed stress

TABLE I.—SUMMARY OF STRESS OBSERVATIONS. TOTAL DEAD AND LIVE LOAD STRESSES IN LB. PER SQ. IN.

	Total Load on Floor, lb. per sq. in.		
	370 Design Load.	535	738 Twice Design Load.
CONCRETE STRESSES.			
On drop at edge of capital:			
Diagonal direction	570	1030	1550
Long cross direction	460	710	700
Short cross direction	240	440	570
Average	420	730	940
On slab at edge of depressed head:			
Diagonal direction	530	900	1140
Long cross direction	760	1380	1600
Short cross direction	270	590	700
Average	520	960	1150
At top of capital:			
Diagonal direction	510	860	910
Long cross direction	610	870	1230
Short cross direction	360	560	1050
Average	490	760	1060
On top of slab at center:			
Diagonal direction	460	870	1310
Long cross direction	550	770	1260
Short cross direction	430	850	1510
Average	480	830	1360
REINFORCEMENT STRESSES.			
Over edge of capital:			
Diagonal direction	3500	5200	10800
Long cross direction	2500	3000	7000
Short cross direction	2100	3000	7300
Average	2700	3700	8400
Over edge of depressed head:			
Diagonal direction	5400	8700	14200
Long cross direction	4100	7800	10400
Short cross direction	2100	2900	5800
Average	3900	6500	10100
At center of span:			
Diagonal direction	3000	6600	12900
Long cross direction	3300	5700	10200
Short cross direction	3100	12300	24200
Average	3100	8200	15800

in the various bands were distributed in each case over the same reinforcement area as was actually present in the diagonal bands at the center of the spans. Tables III, IV and V give the values of the tension in the reinforcement, at certain sections, considering

TABLE II.—SUMMARY OF DEFLECTION OBSERVATIONS.

Load, lb. per sq. ft.	130	130	250	250	415	415	618	618	618	618	0	0	0	See p. 7	25
No. of panels	5	5	5	5	3	3	2	2	2	2					5
Load applied	1 hr.	4½ hrs.	1 hr.	16 hrs.	1 hr.	18 hrs.	1 hr.	25 hrs.	44 hrs.	49 hrs.	17 hrs.	21 hrs.	39 hrs.	6 d. 15 h.	7 d. 15 h.
Gage line															
1	−.0375	−.0395	−.0775	−.0575	−.0725	−.0550	−.0848	.....	−.0635	−.0815	−.0544	.....	.....	.....	.....
2	−.0145	−.0205	−.0237	−.0165	−.0230	−.0180	−.0310	.....	−.0320	−.0040	−.0214	.....	.....	.....	.....
3	−.0162	−.0212	−.0142	−.0008	.0081	.1223	.0322	.....	.0573	.0498	.0103	.....	.....	.....	.....
4	.0362	.0452	.0622	.0742	.0485	.0472	.0312	.....	.0346	.0247	.0534	.....	.....	.....	.....
5	−.0550	−.0740	−.0880	−.0777	−.0960	−.0800	−.1010	.....	−.1126	−.1250	−.1375	.....	.....	.....	.....
6	−.0100	−.0150	−.0085	−.0000	−.0125	−.0040	−.0055	.....	.0075	.0035	.0175	.....	.....	.....	.....
7	−.0067	−.0122	.0098	.0258	−.0264	−.0159	−.0242	.....	−.0124	−.0220	−.0137	.....	.....	.....	.....
8	.0625	.0668	.1308	.1468	.0548	.0508	.0471	.....	.0588	.0508	.0647	.....	.....	.....	.....
9	.0312	.0302	.0742	.0825	.0717	.0782	.1305	.....	.1567	.1522	.0473	.....	.....	.....	.....
10	.0395	.0385	.0920	.1085	.1875	.2045	.3285	.3932	.4139	.4095	.1543	.1446	.1104	.1187	.0933
11	−.0380	−.0500	−.0670	−.0620	−.0720	−.0640	−.0730	.....	−.0830	−.0920	−.0935	.....	.....	.....	.....
12	.0048	.0021	.0208	.0338	.0186	.0288	−.0240	.....	−.0252	−.0352	−.0117	.....	.....	.....	.....
13	.0063	.0038	.0213	.0328	.0808	.0918	.1278	.....	.1550	.1478	.0408	.0403	.0336	.0368	.0313
14	.0413	.0438*	.1140	.1311	.0253	.0258	.0163	.....	.0198	.0103	.0096	.....	.....	.....	.....
15	.0513	.0536*	.1198	.1383	.1318	.1363	.1863	.....	.2156	.2088	.1033	.0861	.0640	.0703	.0444
16	.1219	.0812	.1137	.1347	.2202	.2674	.3592	.3760	.3972	.3687	.0285	.0514	.0368	.0386	.0115
17	.0304	.0344	.0884	.1144	.2254	.2519	.2304	.....	.2588	.2503	.1228	.1201	.0793	.1124	.1063
18	.1292	.1207	.1747	.2010	.2967	.3267	.1582	.....	.1605	.1467	.1370	.1352	.1090	.1174	.1060
19	.0074	.0002	.0682	.0805	.1307	.1472	.0387	.....	.0262	.0127	.0300	.....	.....	.....	.....
20	−.0161	−.0241	−.0171	.0002	−.0061	.0094	−.0211	.....	−.0096	−.0226	−.0044	.....	.....	.....	.....
21	−.0237	−.0292	−.0332	−.0259	−.0407	−.0332	−.0452	.....	−.0492	−.0592	−.0582	.....	.....	.....	.....
22	−.0610	−.0755	−.0760	−.0640	−.0706	−.0495	−.0905	.....	−.0825	−.0950	−.0886	.....	.....	.....	.....
23	−.0160	−.0290	−.0235	−.0240	−.0137	.0020	.0160	.....	.0365	.0330	−.0112	.....	.....	.....	.....
24	−.0613	−.0936*	−.1013	−.0888	−.1298	−.1098	−.1393	.....	−.1666	−.1868	−.2005	.....	.....	.....	.....

* Load added above this point before this reading was taken.

the total tension as observed to be distributed over a fixed reinforcement area. These figures are not actual stresses, and may be used for comparative purposes only.

Tables III, IV and V give comparative reinforcement stresses only. To arrive at a conception of the comparative moments at

TABLE III.—COMPARATIVE STRESSES AT CENTER OF SPAN. TOTAL DEAD AND LIVE LOAD ON FLOOR, LB. PER SQ. IN.

Band.	Loading, lb. per sq. ft.		
	370 Design Load.	535	738
Short cross band	1850	8340	14200
Diagonal band, interior panel	2080	3240	8950
Diagonal band, wall panel	3760	7540	14800
Long cross band, interior panel	3000	2740	8520
Long cross band, wall panel	4770	9380	14300

TABLE IV.—COMPARATIVE STRESSES OVER EDGE OF CAPITAL. TOTAL DEAD AND LIVE LOAD ON FLOOR, LB. PER SQ. IN.

Band.	Loading, lb. per sq. ft.		
	370 Design Load.	535	738
Short cross band	1710	2760	6540
Diagonal band	3620	5600	7880
Long cross band	3000	3540	7000

TABLE V.—COMPARATIVE STRESSES OVER EDGE OF DEPRESSED HEAD. TOTAL DEAD AND LIVE LOAD ON FLOOR, LB. PER SQ. IN.

Band.	Loading, lb. per sq. ft.		
	370 Design Load.	535	738
Short cross band	2000	2760	5500
Diagonal band	5120	8270	13470
Long cross band	4270	9300	13900

the various sections, we must allow for the fact that while the same reinforcement area was considered in each band, the depth from the compressive face of the slab to the reinforcement varied, being about 8 in. at the center, about 14 in. at the column capital, and about 7 in. at the edge of the depressed head. The tension

existing in the concrete of the test floor also exerted its influence on the reinforcement stresses, and this is a variable quantity depending on the amount of load on the floor. The distribution of the load at the various stages must also have had a very appreciable effect especially on the stresses in the long cross bands.

(*e*) *Effect of Drops.*—In the typical flat slab construction where the slab is supported simply by the column capital, the critical stress section is located at the edge of the capital, as is shown by several previous tests for actual stresses. In the Larkin Warehouse, the depressed head design was carried out, which leaves a full thickness of slab (much thicker than other flat slab designers employ) for a considerable distance around the column capital and employs a much thinner slab at the center of the span. The stress readings show that with the proportions used in the Larkin Warehouse, the critical stress section is located at the edge of the depressed head, instead of at the capital although the difference between the stresses at the two points was very slight. The deflection readings, compared with those of other tests, show that the resulting structure is stiffer and stronger than the straight flat slab as ordinarily designed, with anywhere near the same amount of materials.

(*f*) *Other Test Indications.*—Signs of stress were carefully watched and recorded, and were such as to check very satisfactorily the stress measurements. At design load there was practically no visible evidence of stress, and throughout the test the indications were much fainter than has been the case in any building test which the writer is familiar with, including two beam and girder constructions and many flat slab constructions of various designs.

The test has afforded most valuable design data. While the structure showed itself under test to be of excess strength in every respect, the test discloses ways in which the materials may be used more economically in rectangular panels, and indicates the relation between interior and wall panels. The point of inflection readings are most interesting scientific data, both as locating the approximate position of the line of inflection and as giving indications of the presence and magnitude of the arch action.

The writer hopes that the National Association of Cement Users may take up again and carry forward the testing program so

energetically and successfully started two years ago.* While individual manufacturing and selling companies may make many and extensive tests, such as the ones described in this and previous papers† before this Association, the problem is too big for any one or two companies to solve, and it is especially needful that such questions as the location of the line of inflection, the amount of arch action, the effect of openings, the comparison of beam and girder with flat slab construction, and similar problems should be undertaken by scientific experts working under the supervision of an impartial national body like this Association. Business policy limits private investigations, in most cases, to the collection of such data as is necessary for ordinary design purposes to the exclusion of more exact scientific study of the structural action of completed buildings subjected to loads. Private investigations, also, can hardly undertake the financial burden of testing for the effect of repeated loading—a most important matter thus far unattempted in tests for actual stresses in completed buildings. The writer is undertaking in a small way the measurement of stresses under commercial load conditions over long periods of time, but is not ready at present to report any results. This, too, is a line of investigation in which an established organization is needed.

* See *Proceedings*, Vol. VIII, 1912, pp. 61–167.—ED.
† See *Proceedings*, Vol. VII, 1911, pp. 156–179.—ED.

DISCUSSION.

Mr. Robert A. Cummings.—I would like to ask Mr. Condron what moments he used for the outside panels. Mr. Cummings.

Mr. T. L. Condron.—The Chicago Building Code states that the moment in the end panels of beams, girders and slabs shall be figured as $\frac{wl^2}{10}$; intermediate panels $\frac{wl^2}{12}$, where they are continuous. This type of construction is not covered by the Chicago Building Code, because it is not beams and girders, and the Commissioner of Buildings appointed a special commission to prepare formulas for this girderless construction, consisting of Arthur N. Talbot, University of Illinois, W. K. Hatt, Purdue University and L. E. Ritter. They recommended that the end panels be designed for a moment of $\frac{wl^2}{10}$, for wall bearings, or where the end conditions are different, they shall be designed in accordance with the actual end conditions. Wherever we can, we set the columns inside the wall and carry the wall on the slab outside the columns and thus reduce the moment in the end span. Mr. Condron.

Mr. A. E. Lindau.—The actual bending moments in the flat slab occur over the columns, over the support, and the bars that extend into the bottom of the slab in that region most assuredly assist in carrying compression; while the tension at that point may be very problematical, they do assist in carrying something that certainly does exist. It would probably be more profitable to have compression there than just tension. Mr. Lindau.

Mr. Condron.—I agree with Mr. Lindau's statement. There is a region, Mr. President, of reversal of stresses from compression in the bottom to tension in the bottom, and from compression in the top to tension in the top, a region I have referred to as the region of contraflexture, where for various loadings there must be a shifting of stresses. Therefore we prefer to have steel in both top and bottom, throughout that region. We do not consider that any reinforcing passing diagonally through the concrete is effective for tension fiber stress or compression due Mr. Condron.

Mr. Condron. to bending moments, and therefore, we place our tension reinforcement strictly horizontal, where we want to figure it for tension.

Mr. Cummings. MR. CUMMINGS.—What is "circumferential tension?"

Mr. Condron. MR. CONDRON.—Just what the term expresses, and may be resolved into rectangular components or entirely replaced by "radial tension." It may also be used as a term to conjure with.

Mr. Lord. MR. ARTHUR R. LORD.—It seems to me the measurements of deflection are very unsatisfactory as an indication of a stress. Now in the tests I described a few minutes ago the deflection in the wall panel and the interior panel were almost identical, within a few hundredths of an inch, but the stress in the wall panel was sixty per cent greater than that in the interior panel. I think that would indicate that the measurement of the deflection will not give you any indication of what the stress is in the wall panel, and that measurements of stresses are also necessary from other points of view. Measurements of actual stresses are necessary before you get any reliable data regarding strength of construction.

Mr. Condron. MR. CONDRON.—Mr. Lord is absolutely right; the only reason that my company has not made tests of the actual stresses in the concrete and steel is the great expense involved in making them properly. I cannot understand how any slab or panel having a stress sixty per cent greater than another corresponding panel has the same deflection. It does not seem reasonable. I should say there was occasion to study the measurements to see whether a mistake has been made.

Mr. Thompson. MR. SANFORD E. THOMPSON.—I would like to ask Mr. Lord if the test which he has described was one of those made under the auspices of the Reinforced Concrete Committee of this Association.

Mr. Lord. MR. LORD.—The test was made under an agreement with the Concrete Steel Products Company, by observers from the University of Illinois and myself. I think it is highly desirable that this Association should take charge of such tests and we would be glad in the future to make arrangements with the Association to do this. But we felt at the time that we did not care to risk the delay necessary to make such arrangements, and we also understood that the Association was not doing anything this year in this line. I hope it will be possible in the future for

the Association to do something to help us in making tests. I feel sure that the question of repeated load tests of floors, which must be made before we have the finally desired data, for flat slab floors, can be best undertaken by this Association. For ordinary design purposes the many tests already made have reduced flat slab design to a good working basis, but we must have load tests for repeated stresses. Mr. Lord.

I have also started on a small scale another interesting line of experiments which I think this Association should push, that is, the measurements of stresses over long periods of time under actual loading conditions in buildings; that is, to measure the stresses in the building in use, at intervals over a period of years and determine what stresses actually do occur through long periods of time and under the fluctuating loading conditions; to find out the effect of temperature movements, and so forth. I think such things this Association can do better than any individual, and I certainly would welcome its cooperation in such tests as I may be connected with.

MR. CONDRON.—In view of what Mr. Lord said about the tests of deflection between one panel and another, I want to bring out what I stated with reference to the table of panel tests in St. Louis. The end panel deflected less than the interior panel. The end panel was differently designed, being designed for a greater moment, and I was convinced by the deflection there that we have provided for greater moment than actually results in such panels. Mr. Condron.

I still cannot see how to get greater deflection without corresponding stress. It is the stress that causes the deflection and vice versa. I would like also to ask Mr. Lord if in this test, as in one of the previous tests he described, cracks developed in the floor surface or ceiling surface.

MR. LORD.—There were indications of stress, but they were not as large as I observed in previous tests, both in flat slabs and in beam and girder constructions. The design was in accord with the Chicago building ordinance governing cantilever flat slabs, and that is, in my opinion, in some ways, an ultra conservative ordinance; although I am not prepared to say, without looking into the question of temperature stresses, it should not be so. But for a single test load, starting out with a slab not previously Mr. Lord.

Mr. Lord. loaded, the use of this ordinance results in very low stresses. Now when I say the stress in the wall panel was sixty per cent greater than in the interior panel, I do not mean that either of those stresses were very high; they were very low in both of them, and the difference in actual magnitude was not so very great. But the wall panel was, in this building, more heavily reinforced than the interior. We had measurements of the deflection of the wall panel at two different points and they agreed very well.

Mr. Lindau. MR. LINDAU.—Mr. President, I should like to ask Mr. Lord, whether, in the tests which he has recently made, the panels were rectangular, and whether some analysis of the test data has been made to observe the relations between the results in the shorter direction to those in the longer direction, and what in general was the result?

Mr. Lord. MR. LORD.—In my paper is a table which makes that comparison at the center of the span. If we consider, for instance, in each span, our actual stresses at twice design load and actual amount of steel present, we can work out what the stresses would be in a given steel area, so as to get comparative figures, assuming that the same steel area is in each band. At the center of the span the results would be as follows, considering the stress for an equal amount of steel in each direction: for the short cross span, 14,200 lb. per sq. ft.; for the diagonal band 8,950 lb. per sq. ft.; for the diagonal band in the wall panel, 14,800 lb. per sq. ft.; for the long cross band 24 ft. direction, on the interior panel, 8,500 lb. per sq. ft.; for the long cross band in the wall panel, 14,300 lb. per sq. ft. To sum that up, at the center of the span the stress is practically equal in the three directions in the wall panel; but for the interior panel it is very much less in the diagonal and in the longer cross direction, using the same amount of steel. Over the column capital the stresses are very nearly equal in all directions, considering the same amount of steel to be present.

In all of this I neglected entirely the question of the tension existing in the concrete, and that cannot really be negelcted if you are going to consider the moments. There are simply stresses. I have also neglected the difference in the depth of the steel from the compressive face of the slab.

Mr. Condron. MR. CONDRON.—I would like to ask Mr. Lord in this distribution of this load he speaks of on a rectangular panel 24 x 20 ft.

—did each test load cover two panels and did the test cover an area 48 x 20 ft. or 40 x 24 ft.? Mr. Condron.

MR. LORD.—The test load covered an area 48 x 20 ft. Mr. Lord.

MR. CONDRON.—We made tests on two panels 21 x 16 ft. Mr. Condron. The engineer from the building department selected the panels. He picked out both panels, so that we had a loaded area 42 x 16 ft. We got less deflection, I am satisfied, than if the load had been on the 21 x 32 ft. area.

DESIGN OF RIGID FRAMES IN STEEL AND REINFORCED CONCRETE.

By Sanford E. Thompson* and Edward Smulski.†

The object of this paper is to give formulas and diagrams that will simplify the design of rigid construction in steel and reinforced concrete. In reinforced concrete building construction the entire structure is rigid to a certain extent, but for simplicity of design this rigidity is usually not fully taken advantage of. In steel design insufficient attention is often paid to the bending in the columns caused by the rigidity of the connections. Frequently in both building construction and bridge work, as well as in other designs, the bending moment for the beam is taken as an assumed fraction of wl^2, while the bending moments at the ends and in the columns are disregarded entirely. The effect of this is either to make the whole structure inadequate or to make one part stronger at the expense of another.

It is not proposed in this paper to advocate the accurate computations of all the joints in a building several stories high. In ordinary cases the usual arbitrary practice—provided sufficient attention is paid to negative bending moment—gives safe results since the bending in the columns is negligible. Many cases occur, however, in the design of reinforced concrete, where material can be saved and a much better result obtained from an architectural as well as structural standpoint by the adoption of a rigid frame. It is well suited, for example, for girder bridges, machine shops, and power houses, train sheds, market halls, special types of roof and many other structures or parts of structures. It is adapted also for such structures as churches and auditoriums, since any shape may be used to comply with the architectural requirements. In cases like this, the adoption of a rigid frame permits a considerable reduction in sectional area of members and produces a well-balanced design.

* Consulting Engineer, 141 Milk Street, Boston, Mass.
† Designing Engineer, with Sanford E. Thompson.

Rigid frames are not, as such, being used in America to a very great extent, while in Europe they form a standard practice. A partial reason for this lies in the fact that many engineers do not believe in continuity and doubt the effect of the rigidity. They do not hesitate to figure on concrete resisting positive bending moments in the center, neither do they object to using cantilever construction where negative bending moments must be provided for, but they have a religious fear of combining the two so as to allow the concrete to take the negative bending moment at the support as well as the positive bending moment in the center.

Another reason is that the formulas usually given—and there are hardly any published in English—are not in shape for practical use and frighten an engineer who is not of a mathematical turn of mind. To partially remedy the matter, the writers have endeavored to give formulas and diagrams by the use of which the finding of moments for the rigid frames will be scarcely more difficult than using arbitrary fractions of wl^2.

The rigid frame is a statically indeterminate system with one, three, or more indeterminate quantities, depending upon the number and fixity of the supports. The method used in finding the indeterminate quantities is the same in all cases of rigid frames, the difference being only in the method of procedure.

The method of figuring is based on the theory that the deflection of a body under external loads must be such as to make the sum of the work performed by the internal forces a minimum. The theory, commonly termed the "theory of least work," was originated by Castigliano and further developed by Muller-Breslau in his book on Mechanics.

In the present paper two types of design are specifically considered, the one consisting of a horizontal member rigidly attached to two columns with hinged ends, and the other an A-frame, like a roof attached to two similar columns. The formulas and diagrams are adapted both to the simple designs and also the design where cantilevers extend out from the beams.

As stated above, the columns are assumed to be hinged at the bottom. Such a case occurs where the columns rest on or are but slightly imbedded in the ground, or rest upon another part of the structure without rigid reinforcement at the connections. Where the connections at the base of the column are rigid, the

same computations may be used, although the results will err on the safe side.

An illustration of a problem which may be solved is the design of a girder bridge. Instead of computing the girder and the abutments more or less independently, the structure may be considered as one rigid frame, and, knowing the loads upon the bridge and the earth pressure, the stresses in all portions of the structure may be readily computed by the formulas given below and a design established which will require the minimum amount of material, and which, at the same time, will be rationally designed.

Another case where a design of this type can be applied to advantage is in a structure like a machine shop, which consists simply of wall and roof. By designing the joints at the eaves so as to make them rigid instead of providing, in accordance with common practice, a large excess of material so as to be sure to be on the safe side, and instead, even then of having no knowledge whatever of the stresses that occur at the critical points, the dimensions and reinforcement may be fixed by definite computations based on figured stresses.

Many other applications of the analysis will occur to a designer who hitherto has had no means of solving such problems except by guess work or by laborious mathematical computations.

The formulas and the diagrams apply to steel frames as well as reinforced concrete.

In presenting this treatment and recommending it for reinforced concrete structures, the writers appreciate that it implies a construction in which the joints are as carefully designed and constructed as the rest of the members. The use of concrete has already advanced so far in America for high buildings and other large engineering structures, that it is believed fully time to get in line with the practice adopted throughout Europe of making thorough computations according to the laws and principles of mechanics instead of adopting more or less arbitrary and empirical methods.

It is not proposed to present in full the mathematical work entailed in the derivation of the formulas; in fact, in most cases simply the final formulas, which are obtained by the application of the theory of least work, are given. However, to indicate the

procedure for the benefit of mathematical readers, the general method of analysis is first given, followed by the practical formulas for use in design.

THEORY OF LEAST WORK.

Let us suppose that a portion of a curved beam shown in Fig. 1 is subjected to a force, R, not acting in the center of its cross-section. This inclined force may be resolved into two components, one perpendicular to the section, which is the thrust, N, and the other parallel to it, which is the shear, S. The influence of shear on deflection is very slight, so that the eccentric thrust only need be considered in the subsequent discussion.

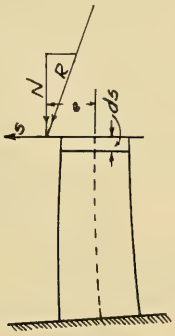


FIG. 1.

A portion of the beam shown is thus subjected to a force, N, acting at a distance, e, from the center, which may be replaced by a force, N, acting at the center and a moment, $M = Ne$.

Consider now a portion of the beam between two parallel sections which are an infinitely small distance apart, ds. The work performed by the eccentric force in deforming this portion may be divided into work done by the force, N, acting centrally and by the moment, $M = Ne$.

Now, let A = area of the section, E = modulus of elasticity, ds = thickness of the portion considered and K^* = work. Then the

* For "work," the letter K has been selected. This term has hitherto been used so little in English that there is no standard letter for it. The German designation, A, however, which stands for *arbeit*, is not adaptable to English treatment because the letter A is used so universally for area.

deformation due to the central forces, N, is $\delta = N/EA\ ds$. In performing this deformation, the force increases from zero to N, and the work, being force times distance, may be found by multiplying the average force, $N/2$, by the deformation:

$$dK_1 = \frac{N}{2}\,\frac{N}{EA}\,ds = \tfrac{1}{2}\,\frac{N^2}{EA}\,ds$$

This represents the work performed by the thrust.

To find the work performed by the bending moment, $M = Ne$, let us assume for the sake of simplicity a rectangular section of homogeneous material (Fig. 2). The stress on a strip $b \times dy$ at a distance, y, from center line of section is $My/I\ bdy$. The deformation in length, ds, due to this stress is $My/IE\ b\ dy\ ds$, and the

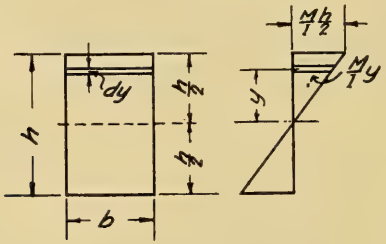


FIG. 2.

corresponding work is $\frac{1}{2}\ M^2y^2/I^2E\ b\ dy\ ds$. The work at any point of the section is thus represented by a function of y. By adding together the work at the different points, which may be done by integration, we obtain the work done on the whole section, or

$$dK_2 = b\int_{-\frac{h}{2}}^{+\frac{h}{2}} \frac{My^2}{2I^2E}\,ds\ dy = \tfrac{1}{2}\,\frac{M^2}{I^2E}\,ds\ b\int_{-\frac{h}{2}}^{+\frac{h}{2}} y^2dy,$$

and since $b\displaystyle\int_{-\frac{h}{2}}^{+\frac{h}{2}} y^2dy = I$, we may write, $dK_2 = \tfrac{1}{2}\,\dfrac{M^2}{IE}\,ds$ as the work performed by the moment.

The work of the eccentric force, that is, the combination of the thrust and moment on the portion of beam, ds, is $dK = dK_1 + dK_2$, and the sum of the work on the whole beam is

$$K = \tfrac{1}{2} \int \frac{N^2}{EA} ds + \tfrac{1}{2} \int \frac{M^2}{EI} ds \quad \ldots\ldots\ldots\ldots (1)$$

The values of N and M are dependent upon the indeterminate quantities, H, V, and M_z.

According to Castigliano, the values of the indeterminate quantities must be such that the work is a minimum. To satisfy this requirement, the first partial derivative of equation (1) in respect to H, V, and M_z, the usual indeterminate quantities must equal zero. This gives the following relation:

$$\frac{\delta K}{\delta H} = o, \quad \frac{\delta K}{\delta V} = o \text{ and } \frac{\delta K}{\delta M_2} = o \quad \ldots\ldots\ldots\ldots (2)$$

from which the values of H, V, and M_z may be determined.

NOTATION.

The following notation corresponds in general to standard practice.

E = modulus of elasticity considered as constant.
I = moment of inertia of columns.
I_1 = moment of inertia of beam.
$\frac{I_1}{I}$ = q = ratio of moment of inertia.
A = area of beam.
h = height of frame.
Z = width of frame.
i = ratio of rise of roof to width of frame.
M = actual bending moment at any point.
M_o = bending moment for a statically determinate system.
N = thrust at any point.
H = horizontal reaction.

GENERAL FORMULA FOR RECTANGULAR FRAME WITH HINGED ENDS.

Before giving the simple formulas for practical use, the derivation of the general formula will be given for the benefit of the mathematical readers.

For frames hinged at the bases of the columns, the horizontal

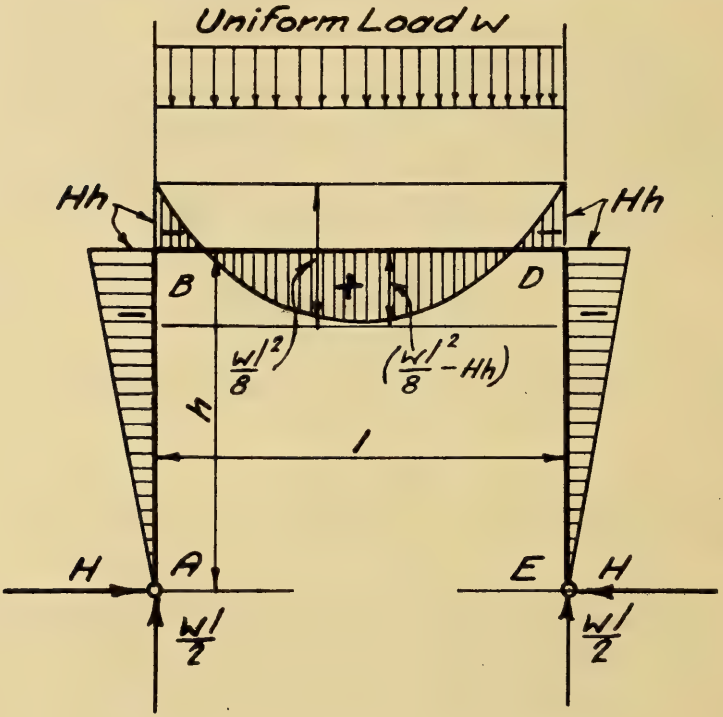


FIG. 3.—RECTANGULAR FRAME WITH UNIFORM LOAD.

reaction, H, is the only statically indeterminate quantity and the following equation is sufficient for determining it:

$$\frac{\delta K}{\delta H} = o = \int \frac{N}{EA}\frac{\delta N}{\delta H}\,ds + \int \frac{M}{EI}\frac{\delta M}{\delta H}\,ds \ldots\ldots\ldots (3)$$

Since E is considered a constant, it will be omitted in the subsequent discussions and formulas.

The values of the moment, M, and the thrust, N, to be substituted in the above must be determined for the different parts of the frame, as shown below.

For columns AB and ED in Fig. (3).

$$N = R_L \quad \text{constant}, \frac{\delta N}{\delta N} = o, \int_o^h \frac{N}{A}\frac{\delta N}{\delta H}\,ds = o$$

$$M = -H_X \quad , \frac{\delta N}{\delta H} = -x, \int_o^h \frac{M}{I}\frac{\delta M}{\delta H}\,ds = \frac{1}{I} H \frac{h^3}{3} \quad (a)$$

For BC in same figure

$$N = -H, \quad \frac{\delta N}{\delta H} = -1, \int_o^h \frac{N}{A}\frac{\delta N}{\delta H}\,ds = \frac{Hl}{A} \ldots\ldots\ldots\ldots (b)$$

$$M = M_O - Hh, \frac{\delta M}{\delta H} = -h, \int_o^h \frac{M}{I}\frac{\delta M}{\delta H}\,ds = H\frac{h^2 l}{I_1} - \frac{h}{I_1}\int_o^l M_O\,dx \quad (c)$$

The integrals above designated as (a), (b), and (c) introduced into equation (3) and solved for H after omitting the negligible quantity, $l/h\ I/A$ give

$$H = \frac{\int_o^l M_O\,dx}{h^2\left(\frac{2}{3}\frac{I_1}{I} + \frac{l}{h}\right)} \ldots\ldots\ldots\ldots (4)$$

The value of $\int_o M_O\,dx$ must be found for every loading.

After determining the horizontal reaction, H, the moments and thrusts at any point of the frame may be easily determined by statics.

WORKING FORMULAS AND DIAGRAMS.

Without going further into the theoretical derivations, the practical side of the problem will be next taken up for the various

types of design selected. As already stated, with the formulas and diagrams given below, it is possible to design all parts of the frames shown in the sketches.

Taking any one of the types of frame, the loads produce bending in all of the members. The bending of the columns produces horizontal reactions at the base (in addition to the vertical reactions). These horizontal reactions are the most difficult forces to determine, but when once found, they, with the known external forces, provide the solution of the entire problem. This is analogous to the problem of a beam in which, when we know the reactions and the external loads, the bending moments can be figured at any point.

The following cases are treated for rectangular frames with hinged bases and for roof-shaped frames with hinged bases:

Uniformly distributed vertical loading.
Concentrated vertical loading.
Loading on cantilever projections.
Horizontal loading such as wind load.
Concentrated horizontal loading.

CASE I. UNIFORMLY DISTRIBUTED VERTICAL LOAD ON RECTANGULAR FRAME (Fig. 3).

The thrust at the base of either column is obtained in the following manner.

Let w = unit load, then

$$\int_o^l M_O \, ds = \frac{wl^3}{12}.$$

Substituting this value in equation (4), and calling

$$C_1 = \frac{\left(\frac{l}{h}\right)^2}{12\left(\frac{2}{3}\frac{I_1}{I} + \frac{l}{h}\right)}$$

we obtain

$$H = C_1 wl. \quad \ldots\ldots\ldots\ldots\ldots\ldots (I)$$

In which C_1 is a constant to be taken from Diagram 1, selected for the proper l/h and I_1/I.

Having thus obtained the value of H, the moments at any

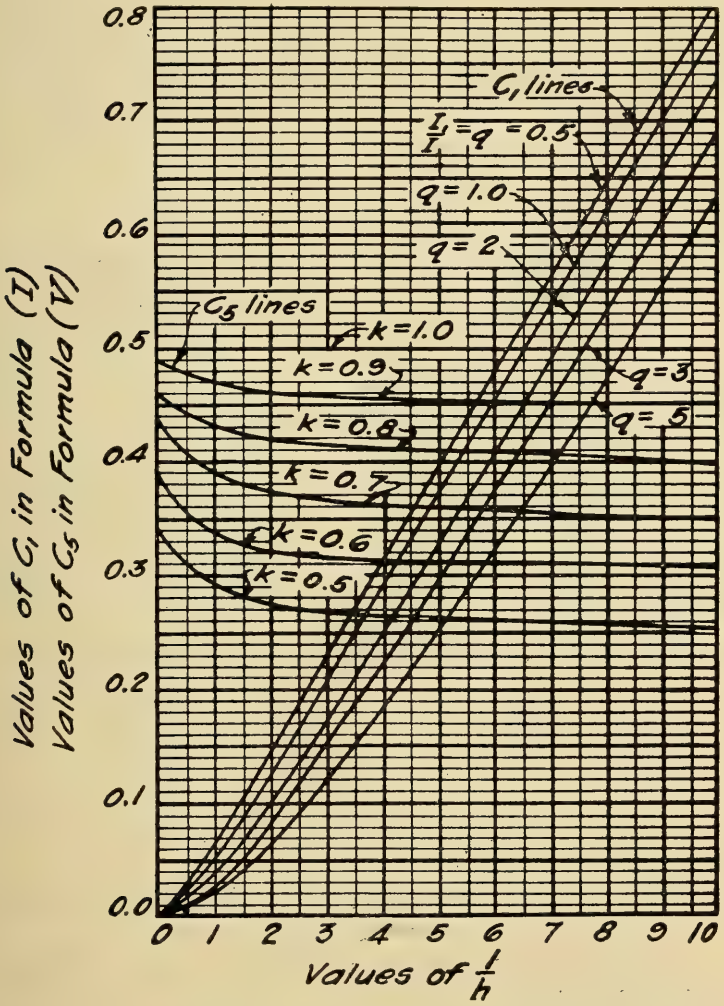


DIAGRAM 1.—VALUES OF C_1 AND C_5.

section in the beam or the column is determined by ordinary principles of mechanics. For example, beam BD is subjected to a negative bending moment at the support, $M_A = M_C = -Hh$,

and a positive bending moment in the middle, $M_{max} = \frac{1}{8} wl^2 Hh$ and also to a thrust $N = H$.

The columns are subjected to a thrust equal to the vertical reaction and a negative bending moment varying from zero at A and E to a maximum of $M = -Hh$ at B and D. The individual bending moments are outlined in Fig. 3, and the combined moment is shown for each member in the cross-hatched area.

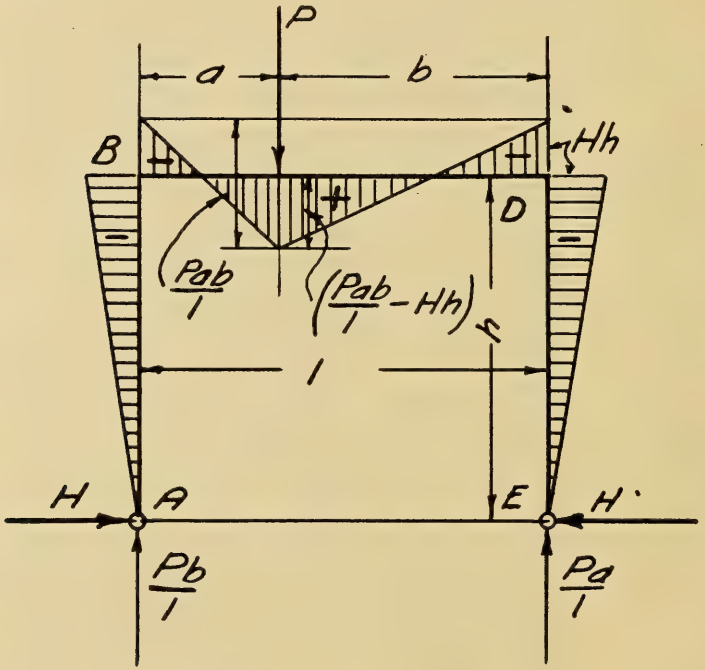


FIG. 4.—RECTANGULAR FRAME WITH CONCENTRATED LOAD.

CASE II. CONCENTRATED VERTICAL LOAD ON RECTANGULAR FRAME (Fig. 4).

The treatment for concentrated load is similar to that for distributed load.

Let P = concentrated load, then

$$\int_0^l Mds = \tfrac{1}{2} Pab.$$

Substituting this in equation (4) and calling

$$C_2 = \frac{\left(\frac{l}{h}\right)^2}{2\left(\frac{2}{3}\frac{I_1}{I} + \frac{l}{h}\right)}$$

we obtain

$$H = C_2 \frac{a}{l}\frac{b}{l} P \quad \ldots\ldots\ldots\ldots\ldots\ldots (II)$$

In which C_2 is a constant to be taken from Diagram 2.

The moments and thrusts at different points are determined as explained in Case I.

CASE III. RECTANGULAR FRAME WITH CANTILEVERS (FIG. 5).

When a frame has projections such as may occur in the roof of a train shed, as shown in Fig. 5, the influence of the load on the cantilever may be determined. The vertical reactions may be found by statics, and the horizontal thrust from the following equation, in which

$$C_3 = \frac{\left(\frac{l}{h}\right)^2}{2\left(\frac{2}{3}\frac{I_1}{I} + \frac{l}{h}\right)} = C_2$$

is

$$H = C_3 \frac{Pa}{l} \quad \ldots\ldots\ldots\ldots\ldots\ldots (III)$$

For values of C_3, see Diagram 2. When both cantilevers are loaded, the horizontal thrust due to the load on the other cantilever must be determined by the above formula and the results added. After determining the horizontal thrust, the moment may be found at any point in the frame.

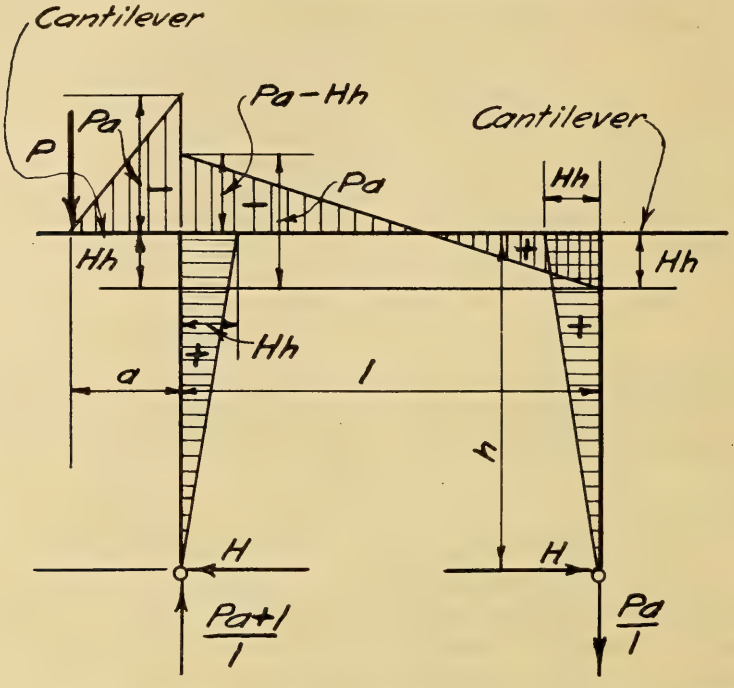


FIG. 5.—RECTANGULAR FRAME WITH CANTILEVERS.

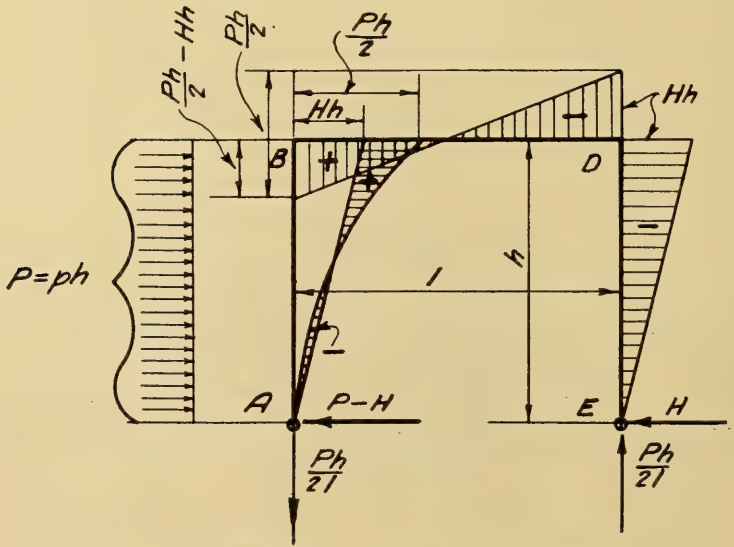


FIG. 6.—RECTANGULAR FRAME WITH UNIFORM WIND LOAD.

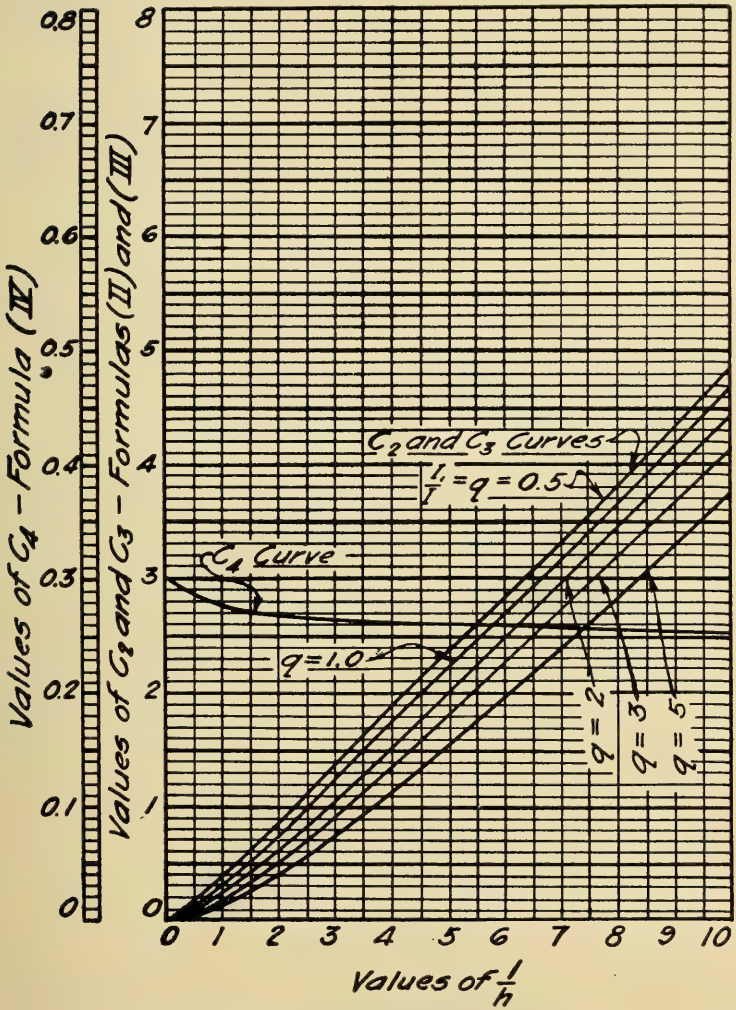

Values of C_4 – Formula (IV)
Values of C_2 and C_3 – Formulas (II) and (III)
C_2 and C_3 Curves
$\frac{I_1}{I} = q = 0.5$
C_1 Curve
$q = 1.0$
$q = 2$
$q = 3$
$q = 5$
Values of $\frac{l}{h}$

DIAGRAM 2.—VALUES OF C_2, C_3 AND C_4.

CASE IV. WIND LOAD ON RECTANGULAR FRAME (Fig. 6).

Let p = the uniformly distributed wind pressure per unit of length, then the total wind pressure is $P = ph$, and the formula for horizontal thrust, in which

$$C_4 = \frac{5\frac{I_1}{I} + 6\frac{l}{h}}{8\left(2\frac{I_1}{I} + 3\frac{l}{h}\right)}$$

is

$$H = C_4 ph \quad \ldots\ldots\ldots\ldots\ldots\ldots \text{(IV)}$$

The value of C_4 may be taken from Diagram 2. These values are the same for all ratios of I_1/I.

The moments at different points in the frame are illustrated in Fig. 6.

The treatment of earth pressure or water pressure is similar except that the horizontal load is not uniformly distributed.

CASE V. CONCENTRATED HORIZONTAL FORCE ON RECTANGULAR FRAME (Fig. 7).

If a force, P, acts horizontally at a distance kh from the base, the reactions are shown in Fig. 7. The vertical reactions may be found by statics, and the value of H, the horizontal thrust, from the following formula, in which

$$C_5 = \frac{3k\left(\frac{I_1}{I}\left(1 - \frac{k^3}{3}\right) + \frac{l}{h}\right)}{2\left(2\frac{I_1}{I} + 3\frac{l}{h}\right)}$$

is

$$H = C_5 P \quad \ldots\ldots\ldots\ldots\ldots\ldots \text{(V)}$$

The value of the constant C_5 is taken from Diagram 1 for different values of k. As the influence of I_1/I is small the curves may be used for any ratio of I_1/I.

The moments are computed in the usual way and are illustrated in Fig. 7.

GENERAL FORMULA FOR FRAME WITH RIDGE ROOF.

A very common type of construction where a rigid frame design may be adopted with economy is that of a sloping roof, such as is shown in Fig. 7. Frequently a building like a picture gallery, an auditorium, a church, or a market, may be of the

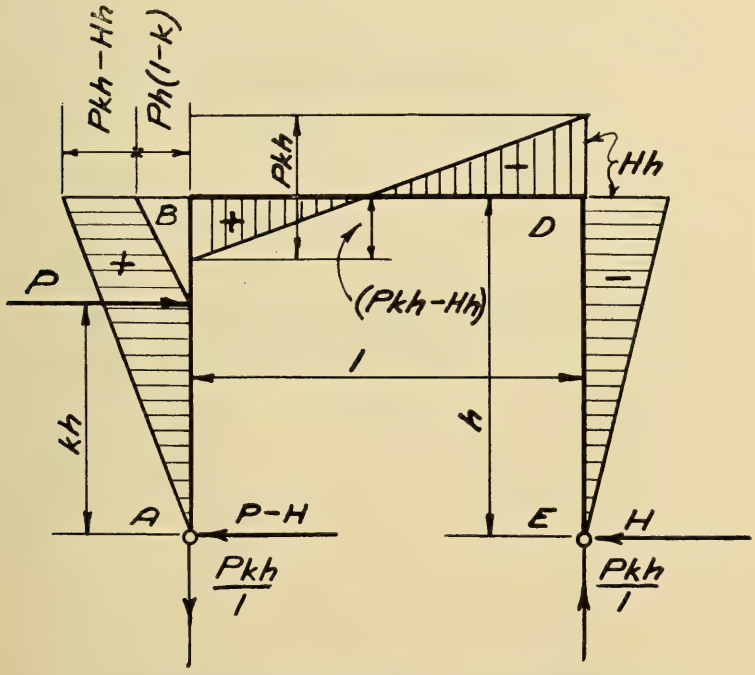


FIG. 7.—RECTANGULAR FRAME WITH CONCENTRATED WIND LOAD.

general type shown, and by adopting rigid frame construction, the horizontal roof chords, frequently so unsightly or obstructive, may be entirely omitted. A machine shop or power house, may have its central part of rigid frame design. These principles of design apply also to arch roofs.

The derivation of the general formulas as similar to that already illustrated for the rectangular frame, although considerably

more complicated. After substituting the solved integrals in the equation (3) we have

$$H = \frac{\int_0^{\frac{l}{2}} M_O\left(h + \frac{i}{2}x\right)dx + \int_{\frac{l}{2}}^{l} M_O\left[h + \frac{i}{2}\left(l - x\right)\right]dx}{h^3\left[\frac{2}{3}\frac{I_1}{I\sqrt{1+\frac{i^2}{4}}} + \frac{l}{h} + \frac{1}{4}i\left(\frac{l}{h}\right)^2 + \frac{1}{48}i^2\left(\frac{l}{h}\right)^3\right]} \quad . . (4)$$

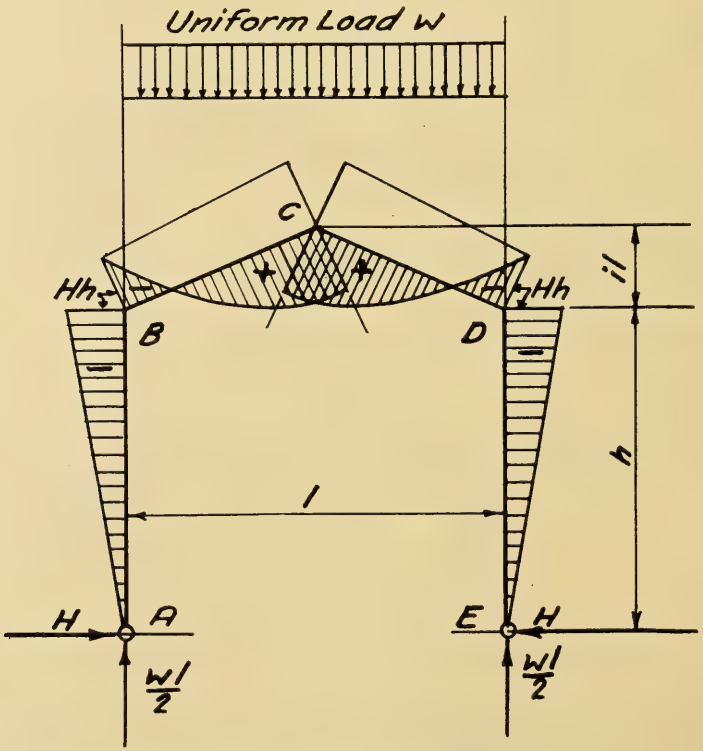


FIG. 8.—ROOF FRAME WITH UNIFORM LOAD.

This equation is altogether too involved for practical use. For special loadings the formulas become simpler and have been used for the compiling of curves so as to make the actual problem

no more complicated than for the rectangular frames already discussed.

Since the type of frame now discussed is adapted chiefly for roofs, it will be termed a roof frame.

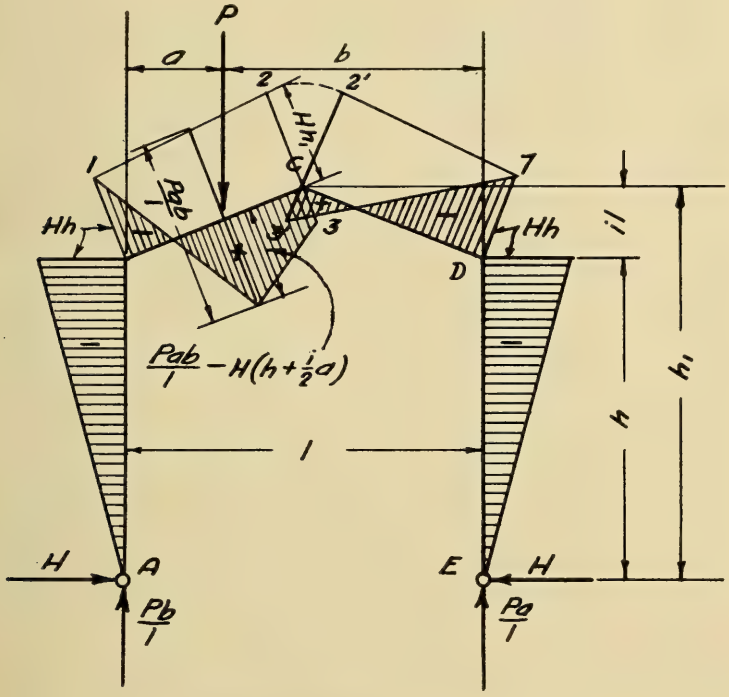


FIG. 9.—ROOF FRAME—CONCENTRATED LOAD.

Area 1234 and 2′3′7 = Positive moments due to $\frac{Pb}{l}$ and P.

Area 1BC2 and 2′CD7 = Negative moments due to H.

Actual moments = shaded areas.

CASE VI. UNIFORMLY DISTRIBUTED LOAD ON ROOF FRAMES (Fig. 8).

In Fig. (8) let w = the uniformly distributed vertical load per horizontal foot on BCD. The horizontal thrust then for a fully loaded roof is

$$H = C_6 wl \ldots\ldots\ldots\ldots\ldots\ldots (VI)$$

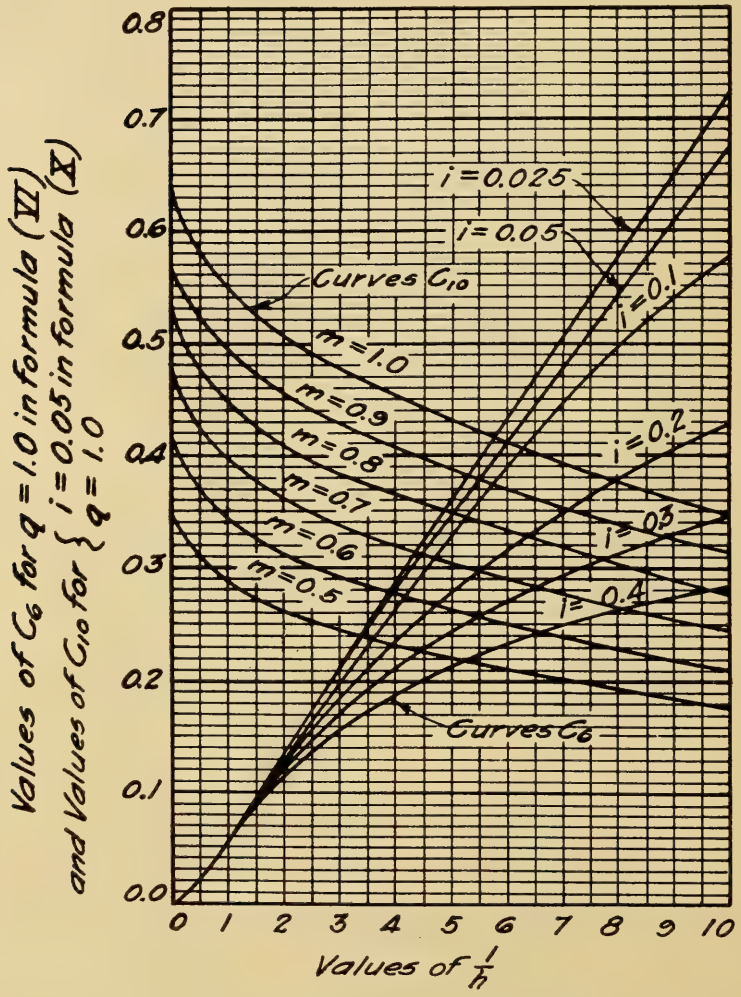


DIAGRAM 3.—VALUES OF C_6 FOR $q=1.0$
AND C_{10} " $\begin{cases} i=0.05 \\ q=1.0 \end{cases}$

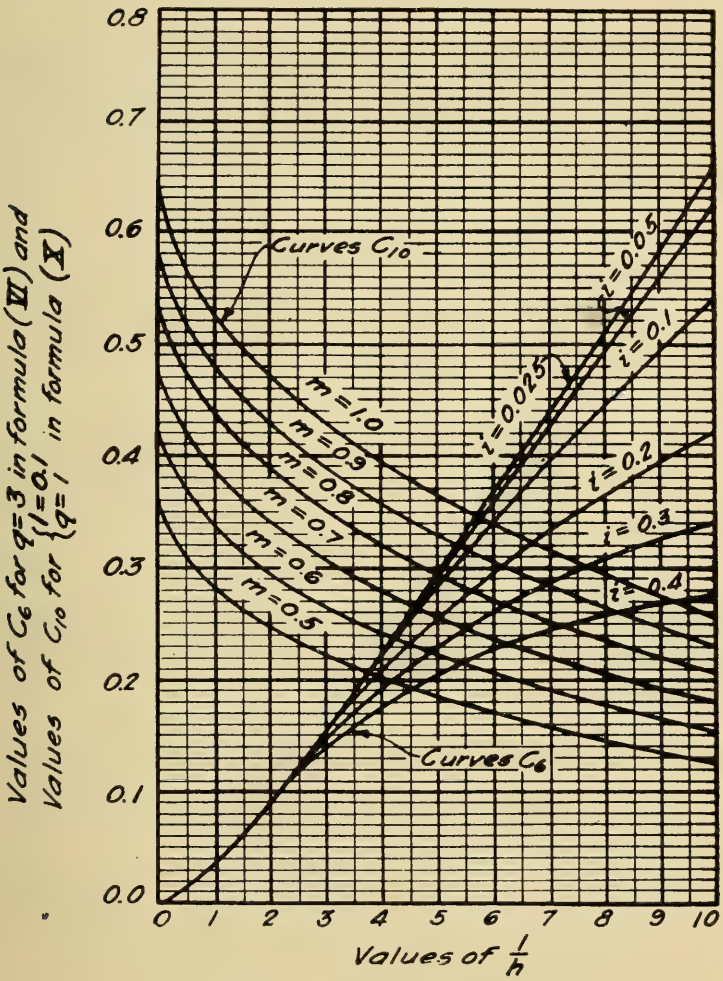


DIAGRAM 4.—VALUES OF C_6 for $q=3.0$
AND C_{10} " $\begin{cases} i=0.1 \\ q=1.0 \end{cases}$

If only half of the roof is loaded, the formula becomes

$$H = \tfrac{1}{2} C_6 wl \ldots\ldots\ldots\ldots\ldots\ldots (VI)$$

The values for C_6 may be taken directly from Diagrams 3 and 4 which are worked out for $I_1/I = q$ equal 1 and 3 and different ratios of i. Occasionally it may be necessary to obtain a value not given in the table, and for this purpose, the constant may be taken as

$$C_6 = \frac{\left(\frac{l}{h}\right)^2\left[1 + \frac{5}{32} i \left(\frac{l}{h}\right)\right]}{12\left[\frac{2}{3}\frac{I_1}{I\sqrt{1+\frac{i^2}{4}}} + \frac{l}{h} + \frac{1}{4} i \left(\frac{l}{h}\right)^2 + \frac{1}{48} i^2 \left(\frac{l}{h}\right)^3\right]}$$

CASE VII. CONCENTRATED LOADS ON ROOF FRAMES (Fig. 9).

The horizontal thrust, H, for a concentrated load at a distance, a, from the support is

$$H = \tfrac{1}{2} C_7 P \ldots\ldots\ldots\ldots\ldots\ldots (VII)$$

Values of C_7 are given in Diagrams 5 and 6 for $q = I_1/I = 1$, values of $i = 0.05$ and $i = 0.1$, different ratios of l/h, and also for ten different positions of load.

CASE VIII. UNIFORM WIND LOAD ON ROOF FRAME (Fig. 10).

The wind is taken as acting on the inclined surface BC. For simplicity, assume that the wind acts horizontally. For any other angle of inclination of the wind, the formulas given below may be used by resolving the inclined force into horizontal and vertical components. Then for the horizontal component, use the formulas given below and for the vertical component follow the plan outlined in Case VI.

Let p = uniform horizontal load per vertical foot, then $P = ilp$, and the thrust may be expressed as

$$H = C_8 P \ldots\ldots\ldots\ldots\ldots\ldots (VIII)$$

Values of the constant C_8 are obtained from Diagram 5.

In case the values are not included in the curves, the thrust may be determined from the formula

$$H = \frac{\frac{1}{3}\frac{I_1}{I\sqrt{1+\frac{1}{4}i^2}} + \frac{1}{2}\frac{l}{h} + \frac{5}{48}i\left(\frac{l}{h}\right)^2}{\frac{2}{3}\frac{I_1}{I\sqrt{1+\frac{1}{4}i^2}} + \frac{l}{h} + \frac{1}{4}i\left(\frac{l}{h}\right)^2} P$$

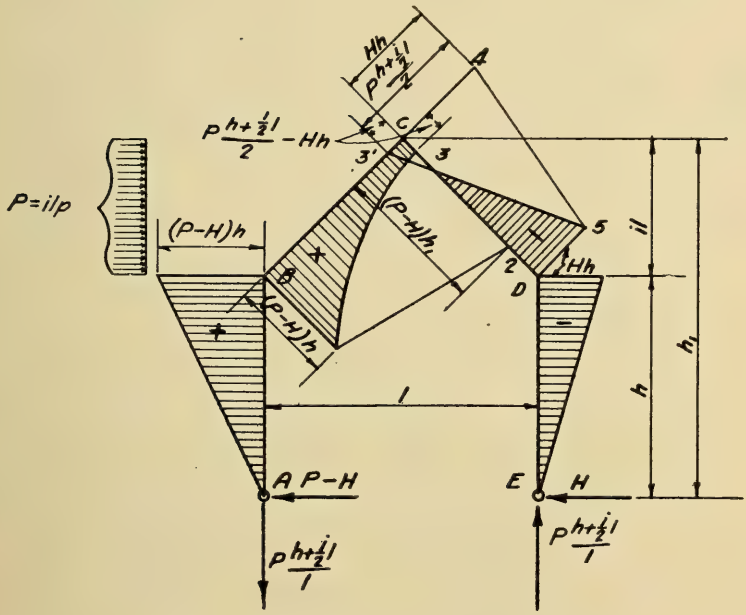


FIG. 10.—ROOF FRAME—WIND ON SLOPE.

Area BC 12 = Positive moments due to $(P\text{-}H)$.

123 = Negative moments due to $P\frac{h+\frac{i}{2}l}{l}$ and uniform wind load.

CD45 = Negative moments due to H.

5 3′ 4′ = Positive moments due to $P\frac{h+\frac{i}{2}l}{l}$.

Actual moments should = shaded area.

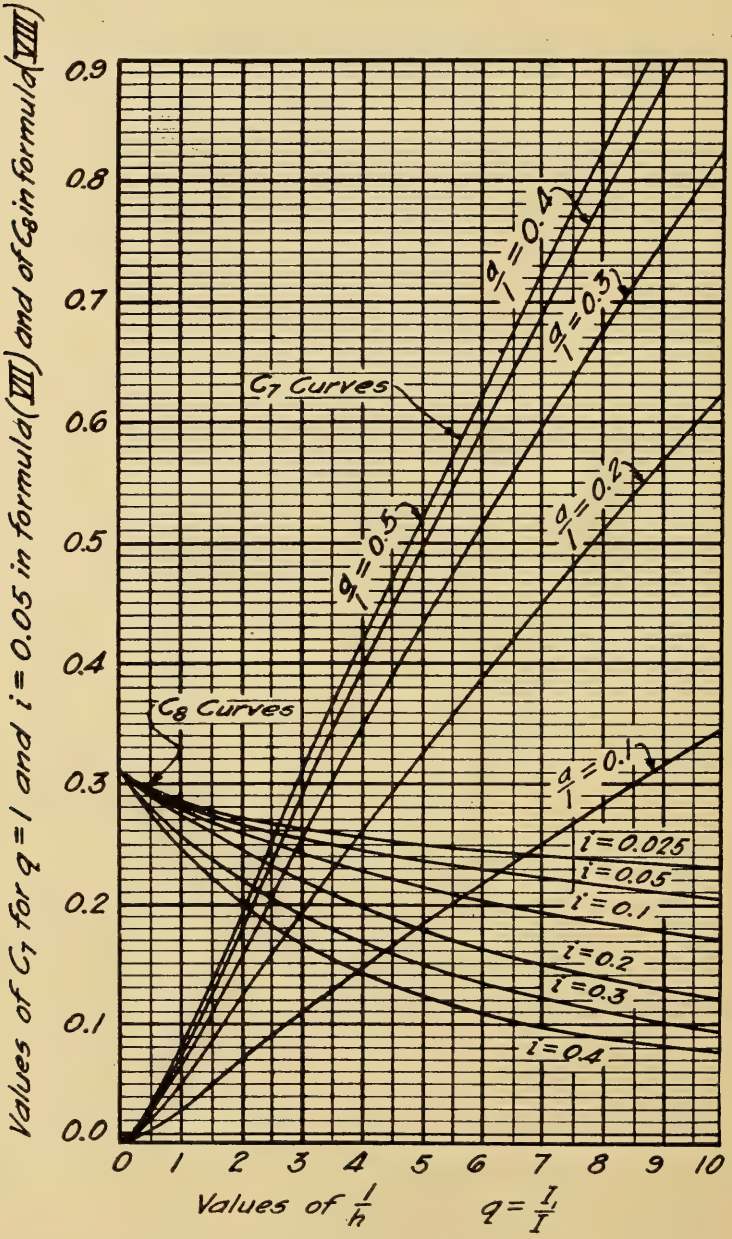


DIAGRAM 5.—VALUES OF C_7 AND C_8.

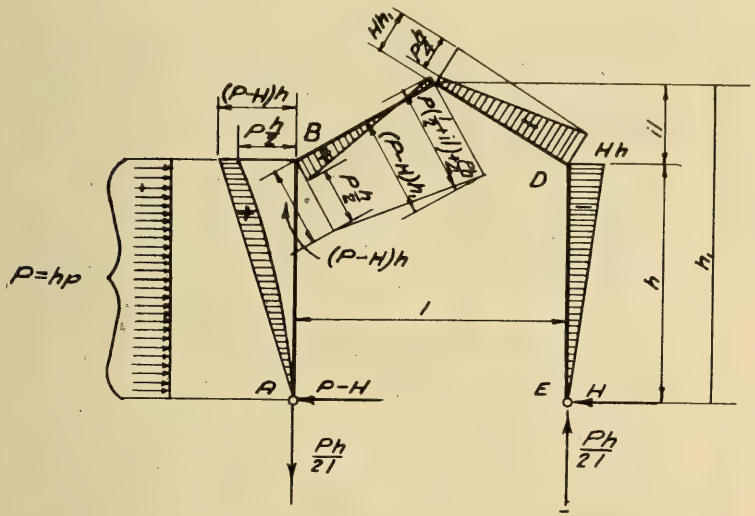


FIG. 11.—ROOF FRAME—UNIFORM WIND LOAD.

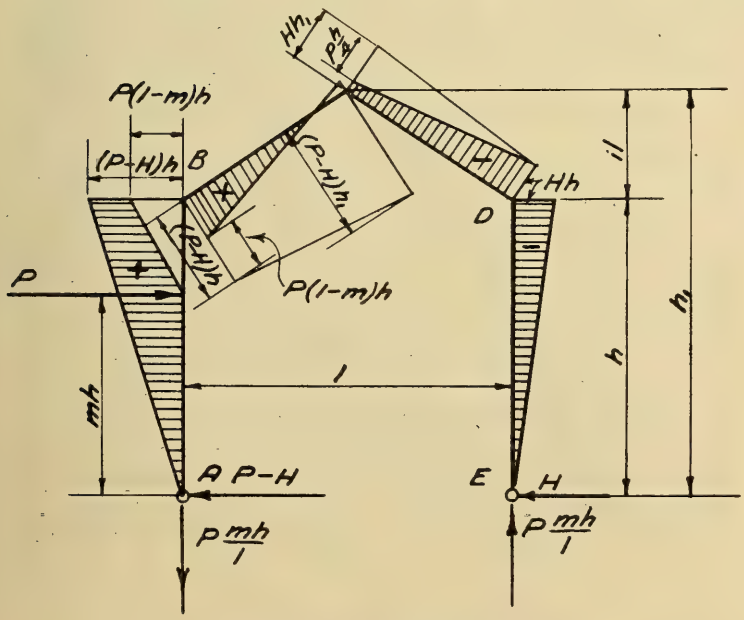


FIG. 12.—ROOF FRAME—CONCENTRATED WIND LOAD.

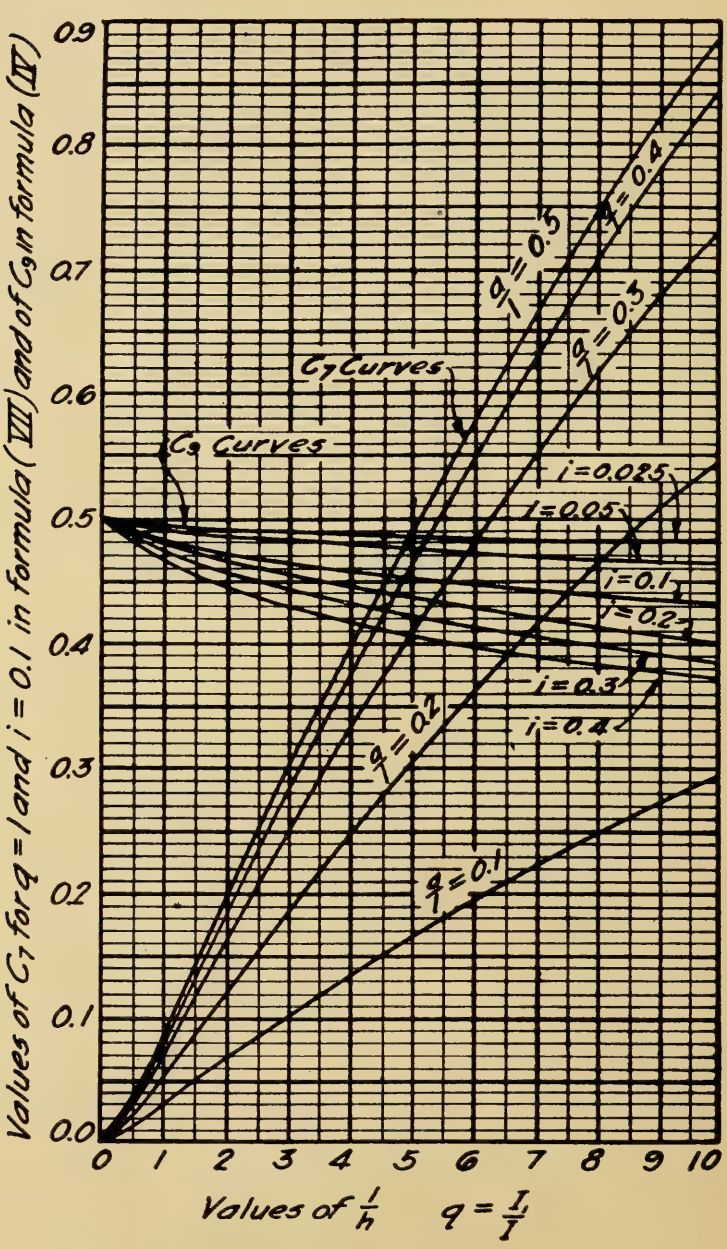


DIAGRAM 6.—VALUES OF C_7 AND C_9.

CASE IX. UNIFORM WIND LOAD ON VERTICAL MEMBER OF ROOF FRAME (Fig. 11).

Let p = uniform load per foot and $P = ph$, then the thrust becomes

$$H = C_9 P \quad \text{.....(IX)}$$

Values of C_9 may be obtained from Diagram 6. In case the values are not included in the curves, the thrust may be determined from the formula

$$H = P \frac{\frac{5}{6}\frac{I_1}{I\sqrt{1+\frac{i^2}{4}}} + \frac{l}{h} + \frac{1}{8}i\left(\frac{l}{h}\right)^2}{4\left[\frac{2}{3}\frac{I_1}{I\sqrt{1+\frac{i^2}{4}}} + \frac{1}{4}i\left(\frac{l}{h}\right)^2 + \frac{1}{48}i^2\left(\frac{l}{h}\right)\right]}$$

CASE X. CONCENTRATED HORIZONTAL FORCE ON VERTICAL MEMBER OF ROOF FRAME (Fig. 12).

For concentrated wind acting at a distance mh from base or for other horizontal force, we have for the thrust

$$H = C_{10} P \quad \text{.....(X)}$$

Values of C_{10} may be obtained from Diagrams 3 and 4.

In case the values are not included in the curves, the thrust may be determined from the formula,

$$H = P \frac{\frac{1}{2}m\left[\frac{I_1}{I\sqrt{1+\frac{i^2}{4}}}\left(1-\frac{1}{6}m^2\right) + \frac{l}{h} + \frac{1}{32}i\left(\frac{l}{h}\right)^2\right]}{\frac{2}{3}\frac{I_1}{I\sqrt{1+\frac{i^2}{4}}} + \frac{l}{h} + \frac{1}{4}i\left(\frac{l}{h}\right)^2 + \frac{1}{48}i^2\left(\frac{l}{h}\right)^3}$$

TESTS TO DETERMINE LATERAL DISTRIBUTION OF STRESSES IN WIDE REINFORCED CONCRETE BEAMS.

By W. A. Slater.*

I. TESTS.

Object of Tests.—It is apparent that in a slab supported on two edges and having a load concentrated on an element g extending in the direction of the span as shown in Fig. 9, the deflection of this element will be greater than that of an element c parallel to it but at some distance from g. The proportion of the total load carried by the various elements will be a function of their relative deflections and also of the curvature of the strip g relative to that of a strip at right angles to it on the center line of the span. An analysis showing the action of such a beam must be based on assumptions which are of doubtful value, even if the conditions of loading and support are accurately known. In practice there will be little uniformity in these respects. The slab forming the cover of a box culvert will be supported on rigid abutment walls and the slab of a beam girder and slab floor system will be supported on beams which deflect at the center causing a larger reaction at the end of the strip c and a smaller one at the end of the strip g than would occur if the supports were rigid. The relative deflections and reactions will also be affected by the properties of the sections involved, by the moduli of elasticity in shear and in tension and by the length, width and depth of the beam considered. Tests of reinforced concrete wide beams indicate that the load distribution is different also at different stages of the test even after failure of the concrete in tension. In view of the complexity of the problem and of the inter-dependence of the variables involved a thoroughly rational analysis to determine the distribution of stresses seems out of the question, though analytical considerations must form a guide for the carrying out

* Engineering Experiment Station, University of Illinois, Urbana, Ill.

of an experimental investigation. Such an investigation has been in progress at the Engineering Experiment Station of the University of Illinois for the past three years. Some of the results of this investigation and the methods of making the investigation form the subject-matter of this paper.

Limitations of the Investigation.—The limitations of such an investigation are numerous. To obtain purely experimental results having any considerable range of applicability it is necessary on account of the large number of variables involved to make a great many more tests than has been practicable so far. It is necessary then in generalizing from the results to use judgment and some speculation as to what would be the effect of varying conditions other than those varied in the tests. For example; the tests so far made have been mostly upon beams of one span and one depth, and of varying widths. To apply the results to beams having other spans and other depths it is necessary to make assumptions as to the manner in which these variables enter.

A fact which limits the progress in tests on reinforced concrete test pieces is the impracticability of profiting by the information gained from one test in designing the next one. All specimens must be tested at the same age, usually 60 days, hence, in order to profit fully by one test in designing the next it must be 60 days between tests and this would be impracticably slow. Therefore, it is necessary to design a whole series of tests at a time. For instance in a series of tests made early in this investigation it was found that many of the beams were failing by diagonal tension and hence were not showing what their strength was against transverse stresses. To guard against diagonal tension failures in the next series of tests, bars were bent up and a weakness in bond strength was introduced which, in the light of tests made in the previous year, was unexpected. Hence although the tests made are believed to be of value, these weaknesses reduce the number of tests from which valuable conclusions may be drawn.

Description and Scope of Tests.—In this paper the two terms longitudinal and transverse are frequently used and refer to directions parallel with and at right angles respectively to the span. The term longitudinal reinforcement is used to mean the reinforce-

ment in the direction of the span even though this be the short dimension of the beam.

Three series of tests on wide reinforced concrete beams have been made at the University of Illinois Engineering Experiment Station. These series were completed in 1910, 1911 and 1912 respectively, and the results formed subject-matter for the bachelor theses of Messrs. K. E. Robinson, E. J. Schell and L. L. Livingston, who graduated in the civil engineering course at the University of Illinois in 1910, 1911 and 1912 respectively.

In making the tests the loads were applied through small rectangular blocks of cast iron or steel embedded in plaster of Paris in order to distribute the load uniformly over the area of the loading block. The centers of the loading blocks in all of the tests lay upon the longitudinal center line of the beam, and in all but two of the tests at the one-third points of the span. An effort was made to provide rigid supports though it is believed in some of the tests this condition was not obtained. Fig. 1 shows the range in width of loads and in width of beams.

In the series of tests of 1910 measurements were taken of deflections at various points of the beam. Fig. 2 shows a beam in the testing machine and the instruments in place for measuring deflections. The frame shown in Fig. 2 is attached to the beam at points on the sides immediately over the supports and at about the height of the neutral surface. Except for the rising of the corners from the supports the deflections are measured from a plane approximately parallel with the position of the neutral surface before testing. Most of these beams had a rigid support but were cushioned on a rubber hose as shown in Fig. 2. In all the tests of the 1911 series and of the 1912 series, measurements were made of deformations in the reinforcement and in some cases of that in the concrete. The measurements were taken in such positions as to show the distribution of the reinforcement stresses laterally from the center. The measurements of deformations in reinforcement and in concrete were made by means of a Berry extensometer (Fig. 3). This instrument and its use have been previously described by the writer.*

The beams of the 1911 series were tested in an inverted position in order to facilitate the taking of deformation measurements.

* See Proceedings, Vol. VIII, 1912, p. 168.

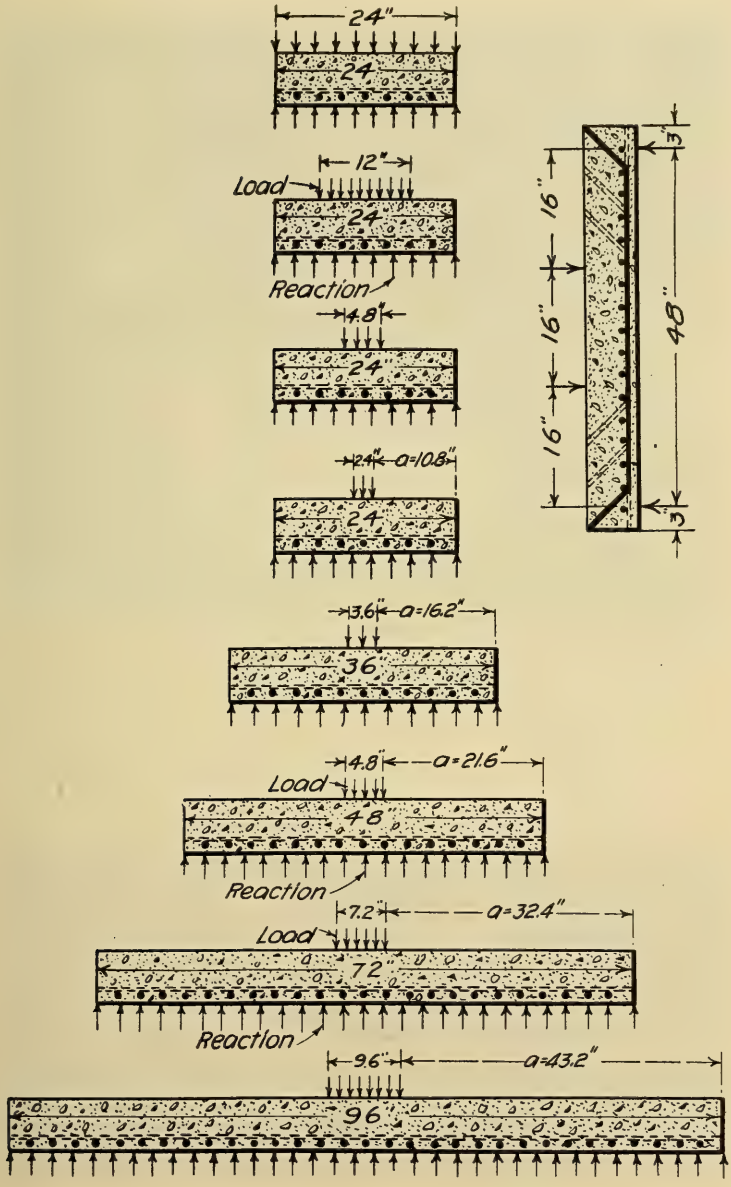


FIG. 1.—WIDTHS OF LOAD FOR BEAMS OF VARIOUS WIDTHS.

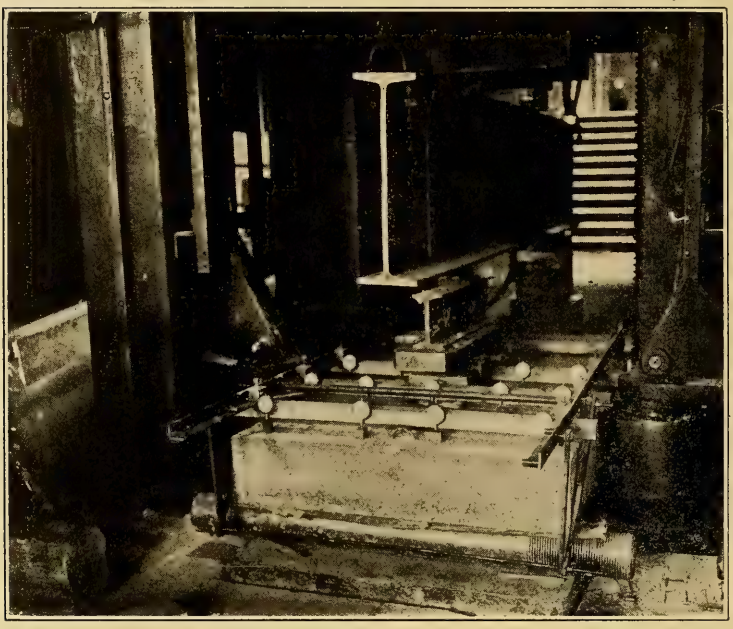


FIG. 2.—WIDE BEAM IN TESTING MACHINE: DEFLECTION INSTRUMENTS IN PLACE; SERIES OF 1910.

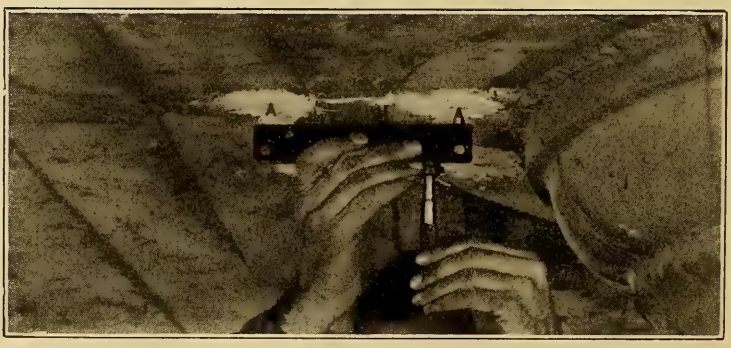


FIG. 3.—THE BERRY EXTENSOMETER.

Fig. 4 shows one of these beams in the testing machine. In these tests the supports consisted of 7-in. and 8-in. I-beams with the I-beams embedded in plaster of Paris at the supports to afford a continuous bearing. The widest beam of this series tested in this way was 36 in., and the indications are that the deflection of the supporting I-beams was negligible. Two beams in this series were quite different from any others which have yet been tested. They were 96 in. wide and had a span of 30 in. The supporting I-beams were put in place when the beams were made, and the concrete was poured around one flange and a portion of the web. This is shown in Fig. 5, a photograph of the testing of one of these beams, and in Fig. 8, a photograph of the beam after testing.

Many of the beams of the 1912 series were wider than could be tested in the testing machines which had been used for the previous tests and the apparatus shown in Fig. 6 was used. The load was applied by means of a hydraulic jack and measured by the nickel steel spring shown in this figure. This spring consists of two nickel steel beams each 2 in. deep, 7 in. wide and 17 in. between centers of supports. The loads were applied at points 9 in. apart and the sum of the deflections of the two beams at the center of the span was measured to about ± 0.0001 in. by means of an Ames gauge-head micrometer set between the beams. The whole weighing apparatus was bolted together firmly so as to form a permanent piece of equipment that could be moved about without impairing its adjustment. Previous to using it in a test it was loaded about ten times sufficiently to stress the steel to approximately 40,000 lb. per sq. in. A deflection of 0.001 in. at the center corresponds to a load of about 1,750 lb. This method of measuring loads was devised and first applied by Mr. M. L. Enger, Assistant Professor of Theoretical and Applied Mechanics at the University of Illinois. The width of most of these beams was so great as to render impracticable the testing of them in an inverted position. Even in the upright position some difficulty was found in securing sufficient rigidity of support to give a satisfactory distribution of the reaction, at the same time leaving the space under the beams sufficiently open for the taking of observations.

In all the beams of the 1910 series of tests the longitudinal

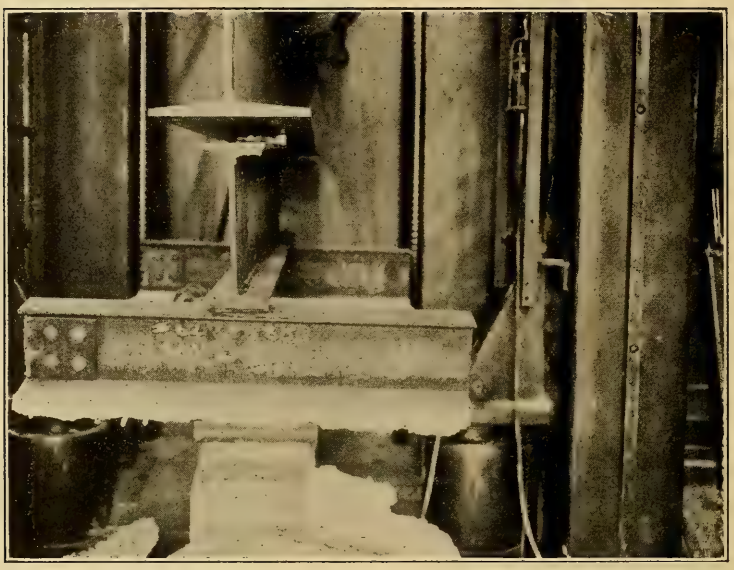


FIG. 4.—WIDE BEAM IN INVERTED POSITION IN TESTING MACHINE; SERIES OF 1911.

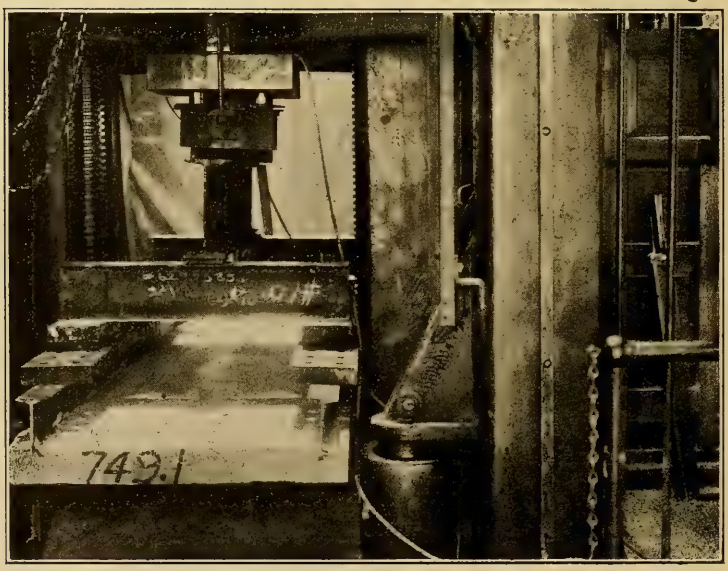


FIG. 5.—TESTING OF 96-IN. BEAM ON SPAN OF 30 IN.; SERIES OF 1911.

reinforcement was about 1.0 per cent. Some of these beams had 0.22 per cent of transverse reinforcement and some had 0.49 per cent, but most of them had none. The depths used were 3 in. and 6 in. and the span of most of the specimens was 48 in. but in a few cases a 72-in. span was used. The beams were 24 in. and 36 in. wide. The width of the load was 0.5, 0.2 and 0.1 times the width of beam, respectively. Most of the attention was placed upon determining what width of load was critical in

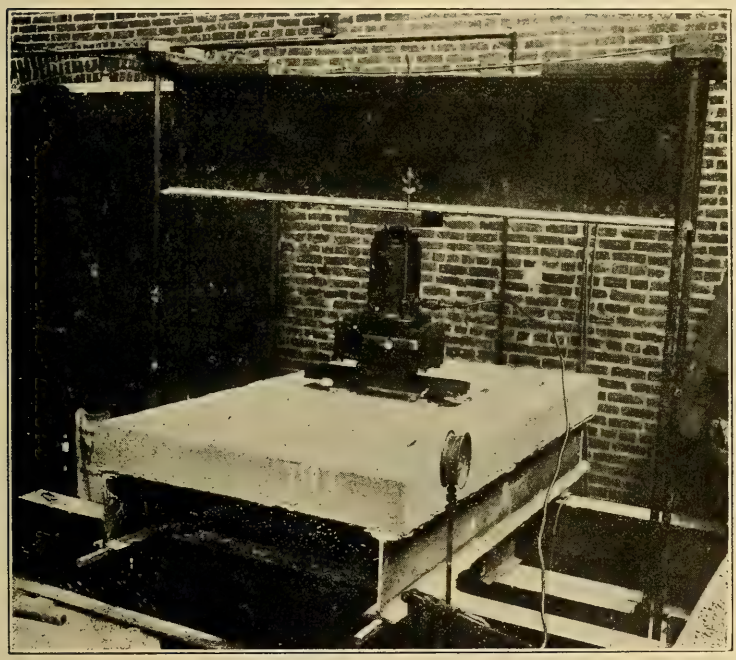


FIG. 6.—TESTING OF 96-IN. BEAM IN HYDRAULIC JACK TESTING MACHINE; SERIES OF 1912.

causing slab failure as defined under *Phenomena of Tests*. It was found that for beams of these widths only the narrowest load used, viz., one-tenth of the width, was at all likely to cause slab failure, and the following year most the loads were concentrated on one-tenth of the beam width, the same type of beam being used. In the 1912 tests the span was still kept at 48 in. and the width varied from 24 in. to 96 in. Two values of longitudinal reinforce-

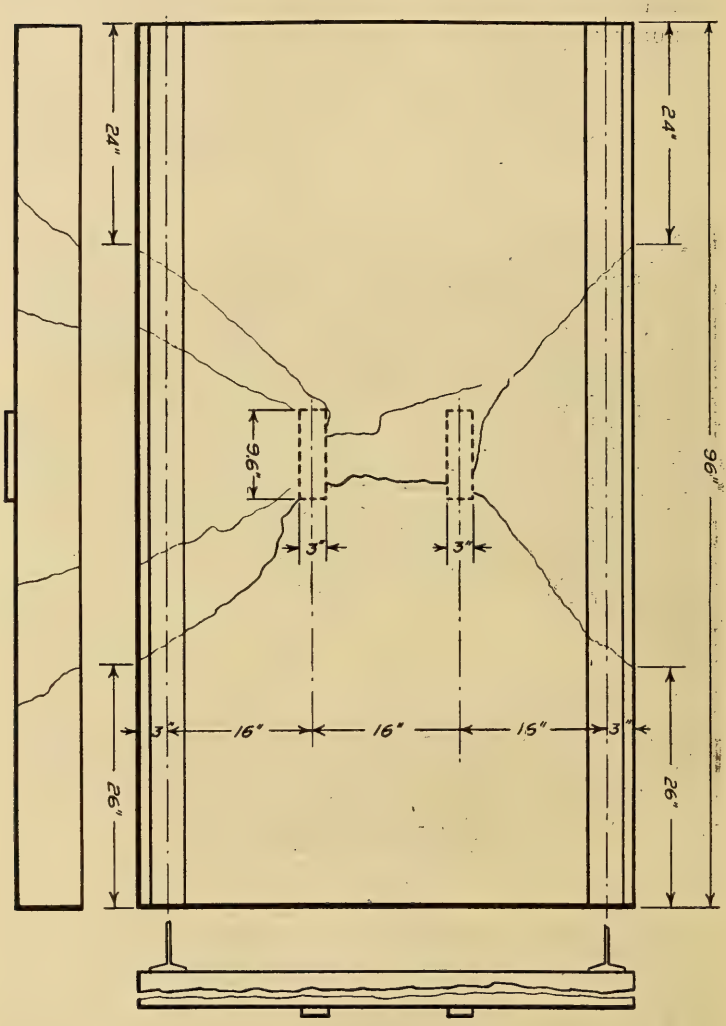


FIG. 7.—CRACKS WHICH ACCOMPANY SLAB FAILURE.

ment were used, 1.0 per cent and 1.5 per cent. The transverse reinforcement was 0.23 per cent, 0.46 per cent and 0.75 per cent respectively. To obviate diagonal tension failure, longitudinal rods were bent up at various places as shown in Fig. 1.

Phenomena of Tests.—In some of the tests a tendency of the corners to curl up has been noted. This was more pronounced in the 1910 series of tests than in any since that time. It is thought this may have been due partially to the fact that most of these beams were cushioned by means of pieces of rubber hose placed between the beam and the rigid support. The natural tendency for the reaction to be concentrated near the central part of the end of the beam as shown in Fig. 1, would result in a greater compression of the hose at this point and a consequent curling up

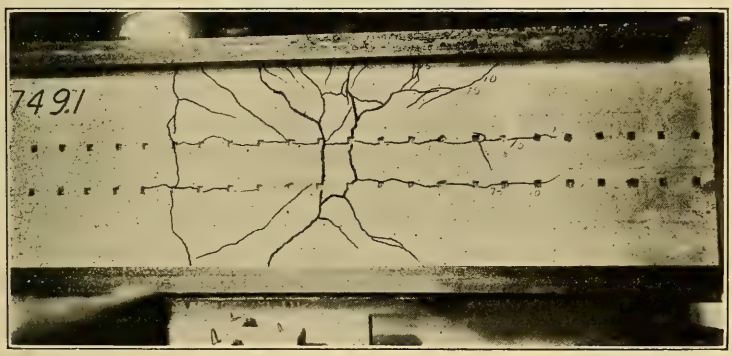


FIG. 8.—CRACKS ON BOTTOM OF VERY WIDE BEAM AFTER FAILURE.

of the corners. In the 1911 and 1912 tests no measurements of deflections were made, but the embedment was in plaster of Paris in all cases, and a marked tendency to curl would have disclosed cracking of the plaster of Paris cushion. This was seldom observed except at loads which gave high stresses in the central longitudinal rods. When this curling did occur it frequently was accompanied by cracks on the ends of the beams as shown in Fig. 7. These cracks were the continuation of a longitudinal crack which formed on the bottom, extending from load point to load point, there forking and extending to the end of the beam at distances from the end varying with the width of the beam. This crack is sketched in Fig. 7. Fig. 8 is a photograph showing characteristic cracks on the tension side of a very wide beam.

In these tests, failure has occurred in four different ways, (1) tension in longitudinal reinforcement, (2) diagonal tension, (3) bond, and (4) transverse tension or failure as a slab. The first three manners of failure are familiar and need no comment, but the fourth is peculiar to this form of test piece and method of loading, hence needs definition. When the tendency of the central element of the beam to deflect more than the elements at the edges, (and hence to cause curvature in a vertical plane at right angles to the direction of the span) is sufficient to cause failure, this will be known as transverse tension failure or as slab failure. In beams having no transverse reinforcement such failure usually follows very closely upon the formation of the longitudinal cracks shown in Figs. 7 and 8. In beams having transverse reinforcement the formation of the longitudinal crack is not necessarily followed by failure as a slab. If the transverse reinforcement passes its yield point before the longitudinal reinforcement does, or before slip of bars occurs, failure is attributed to transverse tension. This, of course, can be exactly determined only by extensometer measurements. Although punching shear has not been mentioned as a primary cause of failure, diagonal tension caused by it often appeared as a secondary phenomenon and may have been of importance in producing bond failure. When present, the fracture due to this cause usually formed an angle of about 45 deg. with the vertical and sloped away from the loading blocks on all four sides.

Analysis.—The following is given not as exact analysis but rather as an illustration showing the nature of the problems involved.

Fig. 9 represents a wide beam of span s and width b supported on I-beams which may deflect, and loaded uniformly on the longitudinal element along the center-line of the beam. Assume that the deflection at the center of the supporting I-beam f will throw just enough of the pressure from the test piece toward the ends of f to produce a uniform bearing on the test piece as shown in this figure. The reactions as shown will cause a moment in a transverse direction and one in a longitudinal direction. Calling the moment at the center of the span in the longitudinal direction M_L and that at the center of the width in the transverse direction

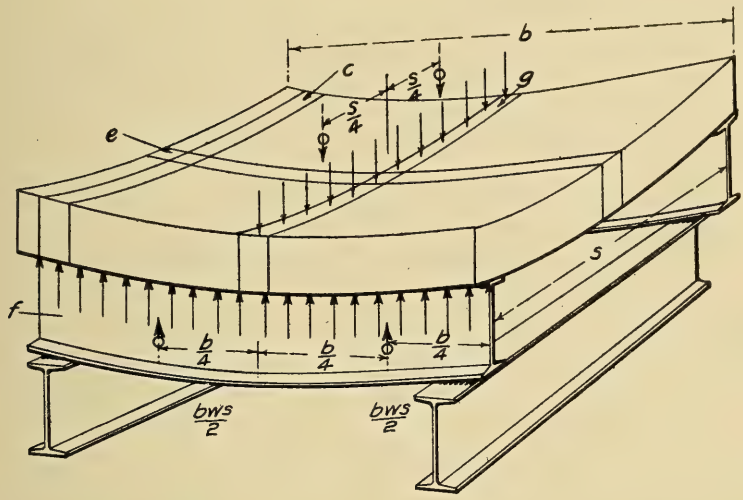


FIG. 9.—SKETCH OF WIDE BEAM WITH LOAD APPLIED UPON CENTRAL ELEMENT.

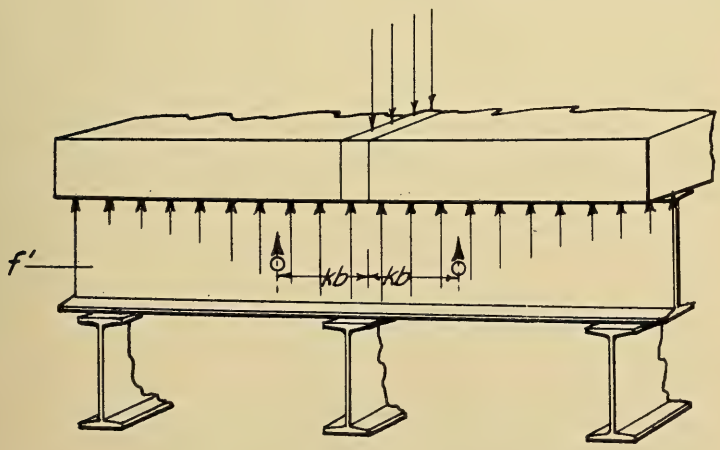


FIG. 10.—SKETCH SHOWING ASSUMED DISTRIBUTION OF REACTION.

M_T and w the load per unit of area of the entire beam, the following relations are derived:

$$M_L = \tfrac{1}{2} wbs\frac{s}{4} = \frac{wbs^2}{8} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (1)$$

$$M_T = \tfrac{1}{2} wbs\frac{b}{4} = \frac{wb^2s}{8} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (2)$$

The intensity of the transverse moment is greater, across the center of the longitudinal span than near the support, but there is nothing in this analysis to bring out this fact.

For a square beam $b=s$ and $M_T=M_L$, and the same total amount of transverse reinforcement would be required as of longitudinal. However, it is apparent that the load would never be distributed in this way. This case was assumed not because it was thought to be representative of what one would expect to find in practice or even in tests, but to show by a concrete illustration the character of what is meant by transverse beam action, and to choose that illustration at one end of the extreme possible range in conditions. A uniformly distributed reaction would never occur in practice and in tests it could be brought about only by special design. The reaction probably would be somewhat concentrated near the center of the line of support as shown in Fig. 10 and the center of gravity of that portion of a reaction coming upon either half of a support would be at a distance kb from the center of the I-beam f, in which k is likely to be less than $\frac{1}{4}$. Then M_T might be expected to vary from the value given in equation (2) in proportion as k varies from $\frac{1}{4}$. It most likely would always be less than the amount shown by the equation. If now the width of the loaded strip becomes finite the load causing transverse moment will no longer be wbs but will be a function of the overhang, a (the distance from the edge of the loading block to the edge of the beam). The distance to the center of gravity of this load will also be a function of a. A consideration of the data in hand and of various possible distributions of the load causing transverse moment indicates that this load and the distance to its center of gravity may without much error be considered to be proportional to a; then

$$M_T = k_1 was \;.\; k_2 a = k_1 k_2\, wa^2s = Kwa^2s \quad \ldots\ldots\ldots\ldots\ldots (3)$$

where k_1k_2 and K are constants.

II. RESULTS OF INVESTIGATION.

Beams without Transverse Reinforcement.—In the beams tested, the stress in the longitudinal reinforcement lying along the center line of the beam was much greater than that in the reinforcement near the edge for beams having a width of 48 in. or more, but for widths less than this the stresses were nearly equal across the width of the beam or very irregular in their variation. Figs. 11 to 18 give the load-stress curves and curves showing the variation in longitudinal stress across the width of beams for 24, 36, 48 and 72-in. beams having no transverse reinforcement. In the latter diagrams, termed stress-distribution diagrams, any plotted point indicates the stress measured in the longitudinal rod which is shown in the beam elevation as lying directly under the plotted point.

It will be noted that the shape of the stress curves corresponds to some extent with the reaction curve indicated in Fig. 10 and there is some justification for the assumption that the stress-distribution curves of Figs. 11, 13, 15 and 17 may be used as load curves of a beam whose span is in a transverse direction and whose reaction begins at the longitudinal section through the edge of the loading blocks. The basis for this assumption is here given. In the tests the load was applied at the one-third point of the span upon a central element of the beam. If all the longitudinal elements had their loads applied at the one-third points of the span the loads carried by the various strips would be exactly proportional to the stress at the center of the span. However, it is clear that the load will not travel directly outward so as to be applied at the one-third points of the span of each strip. In fact it is likely that the shear between the edge of the element under the loading block and the element adjacent to it modifies the distribution of load in both elements, giving it more nearly the effect of a uniformly applied load than it would otherwise have. If the variation in load distribution should be from third point loading on the central element to uniform distribution on the element at the edge of the beam, the variation in proportionality of loads to stresses would be 25 per cent and it seems probable that at least up to the time of tension failure in the concrete along a longitudinal line, the variation in load distribution is less than that indicated. If, then, the assumption be used that

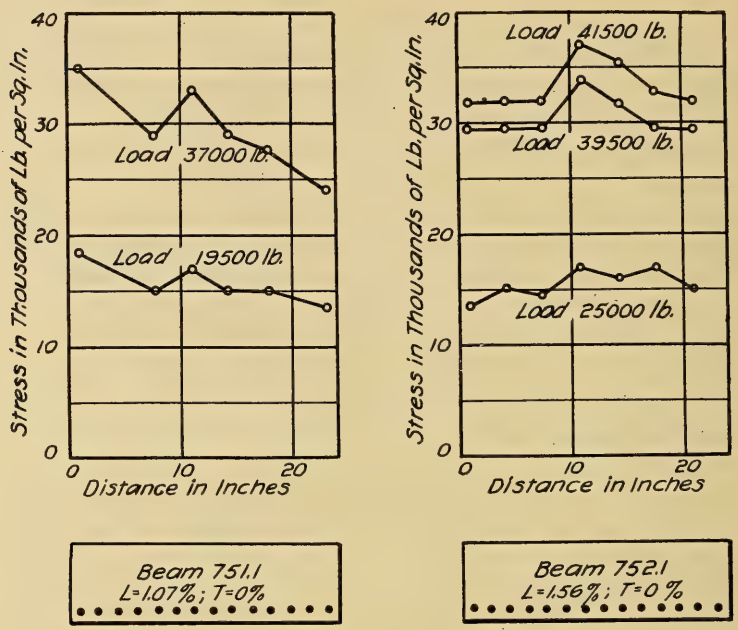


FIG. 11.—STRESS DISTRIBUTION DIAGRAMS FOR 24-IN. BEAMS WITHOUT REINFORCEMENT.

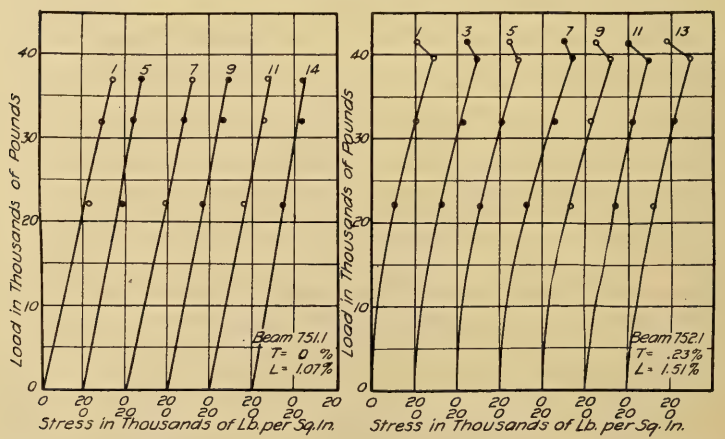


FIG. 12.—LOAD-STRESS DIAGRAMS FOR 24-IN. BEAMS WITHOUT TRANSVERSE REINFORCEMENT.

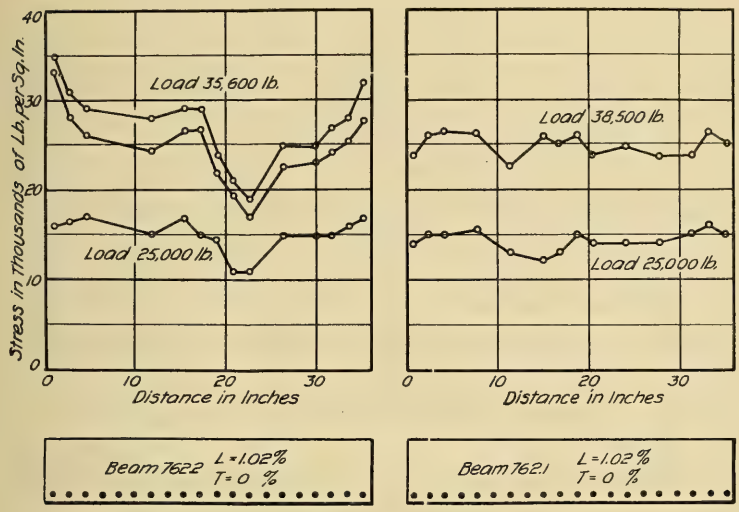

Stress in Thousands of Lb. per Sq. In.
Load 35,600 lb.
Load 25,000 lb.
Distance in Inches
Load 38,500 lb.
Load 25,000 lb.
Distance in Inches
Beam 762.2 L = 1.02 % T = 0 %
Beam 762.1 L = 1.02 % T = 0 %

FIG. 13.—STRESS DISTRIBUTION DIAGRAMS FOR 36-IN. BEAMS WITHOUT TRANSVERSE REINFORCEMENT.

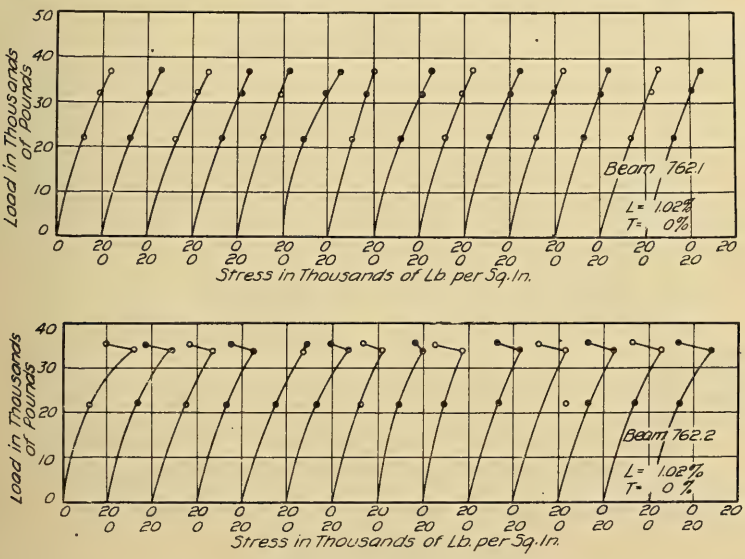

Load in Thousands of Pounds
Beam 762.1
L = 1.02%
T = 0%
Stress in Thousands of Lb per Sq. In.
Load in Thousands of Pounds
Beam 762.2
L = 1.02%
T = 0 %
Stress in Thousands of Lb. per Sq. In.

FIG. 14.—LOAD-STRESS DIAGRAMS FOR 36-IN. BEAMS WITHOUT TRANSVERSE REINFORCEMENT.

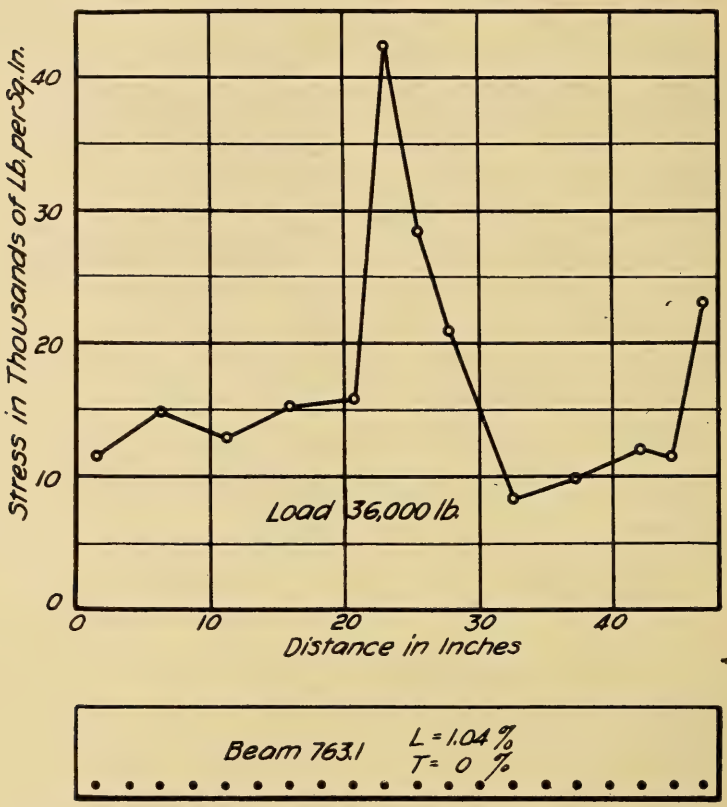


FIG. 15.—STRESS DISTRIBUTION DIAGRAMS FOR 48-IN. BEAMS WITHOUT TRANSVERSE REINFORCEMENT.

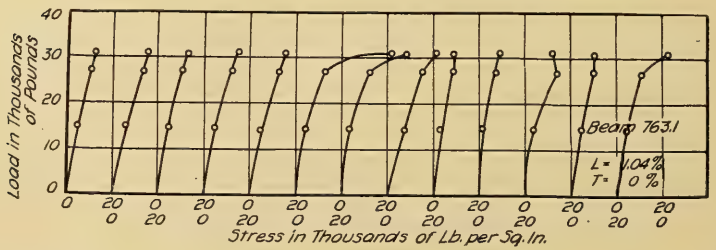


FIG. 16.—LOAD-STRESS DIAGRAMS FOR 48-IN. BEAMS WITHOUT TRANSVERSE REINFORCEMENT.

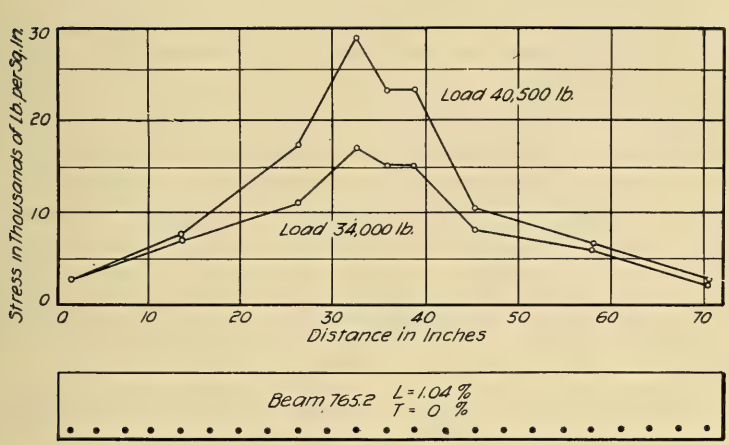


FIG. 17.—STRESS DISTRIBUTION DIAGRAMS FOR 72-IN. BEAMS WITHOUT TRANSVERSE REINFORCEMENT.

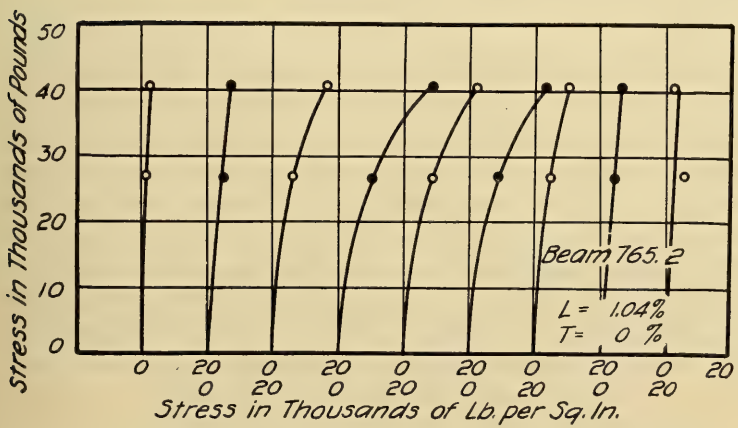


FIG. 18.—LOAD STRESS DIAGRAMS FOR 72-IN. TRANSVERSE BEAM WITHOUT REINFORCEMENT.

the loads on the longitudinal elements are proportional to the measured stresses in them the transverse moments calculated for the longitudinal section along the edge of the loading blocks for a load of 1,333 lb. per sq. ft. on all the beams considered under the present head are as shown in Fig. 19. It may be seen from equation (3), that if M_T be plotted as ordinates of a curve in which the abscissas are values of overhang the curve will be a parabola if the form of the equations is correct. A parabola which fits the results fairly well is shown by the dotted line in Fig. 19. In Fig. 20 the distance, X, to the center of gravity of the assumed load curves are plotted as ordinates and the width of beams as abscissas. In this figure the ratios of ordinates to

TABLE I.—MODULUS OF RUPTURE FOR BEAMS WITH NO TRANSVERSE REINFORCEMENT, LB. PER SQ. IN.

	No. of Tests.	Average.	Maximum Variation from Average, per cent.
24-inch beams	3	253	34
36-inch beams	1	316	..
48-inch beams	4	365	26
72-inch beams	2	371	1

corresponding abscissas will be values of k. For a constant k the curve would be a straight line. The straight line shown in the figure gives the value of ka which if multiplied by a constant ratio of the load on the beam will give the parabola. The curve through the plotted points indicates that k decreases as b increases. The straight line gives 0.23 as a mean value of k for the beams with no transverse reinforcement. If for the maximum load carried, the transverse moment at a longitudinal section along the edge of the loading blocks be calculated by the method outlined above the values of the modulus of rupture obtained are as given in Table I. In this table the effective width is taken as the full length of the beam, and the values given are averages.

The variation in values for similar test pieces is in some cases large but not larger than in other kinds of test pieces where there is so much chance for eccentricity in loading. The agreement

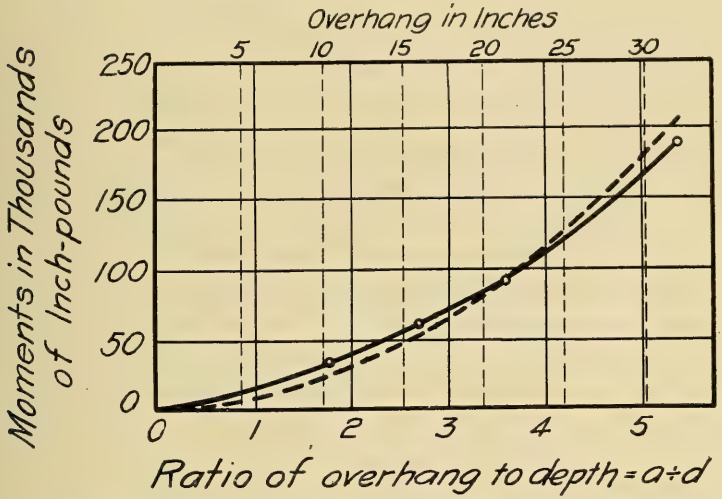


FIG. 19.—TRANSVERSE MOMENTS IN BEAMS WITHOUT TRANSVERSE REINFORCEMENT.

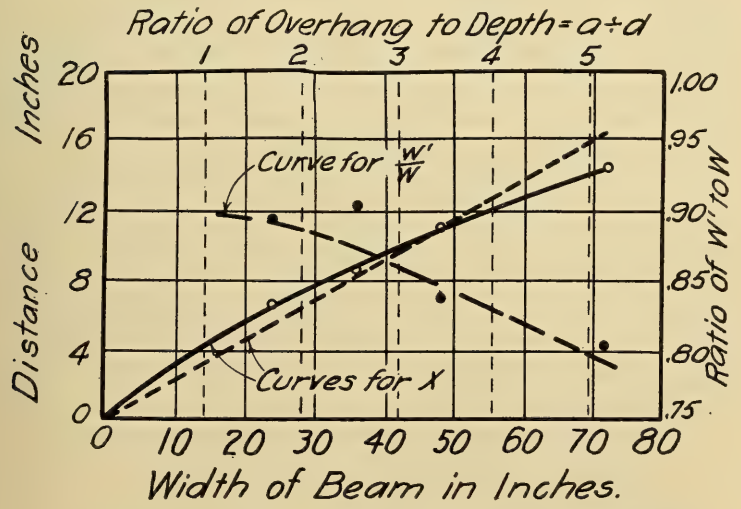


FIG. 20.—RELATIONS BETWEEN WIDTHS OF BEAMS AND LOAD DISTRIBUTION.

X curves: distance from edge of loading blocks to center of gravity of load outside of load blocks. $W' \div W$ curves: ratio of load outside of loading blocks to total load on beam.

between values for different widths is sufficient to indicate that there is some basis for the method used.

The actual value of the modulus of rupture as calculated is not far from that which we might expect to obtain. The values for the 24-in. and 36-in. beams are low. As noted below, the extent to which slab action affected these results is uncertain. It is true that at the load causing failure in these narrower beams a high tensile stress in the steel was present, and except for this weakness it is possible that they would have developed as high a modulus of rupture in the transverse direction as did the wider beams. The use made here of the data is allowable only on the basis that failure was by transverse tension in the concrete. This was true in all the 72-in. and 48-in. beams having no transverse reinforcement. In the 36-in. and the 24-in. beams there is some room for question, but in even the 24-in. beams the variation in stress across the width of the beam shown in Fig. 12 indicates the probability that although in beam 752.1 a high average steel stress was developed, failure as a slab may have been imminent.

Transverse Moments in Beams Having Transverse Reinforcement.—An effort was made to apply to the beams having transverse reinforcement the same methods of calculating transverse moments as has been used in the beams having no transverse steel. That is, the loads carried by longitudinal elements were assumed to be proportional to the steel stress in the element. On this basis the transverse moments in beams having 0.23 per cent transverse steel (Figs. 21 to 26, inclusive) were calculated for a section at the edge of the loading block. Using the moments thus calculated to determine stresses in the transverse steel, results were obtained which are far beyond the ultimate strength of the steel. The actual resisting moment at this section was then caculated by taking the summation of the products of the measured stresses, the steel areas and the arm of the resisting couple (assumed to be about 0.87 of the depth to the transverse steel). The values thus obtained for a load of 1,875 lb. per sq. ft. plotted against values of the ratio $\frac{a}{d}$ form the lower curve in Fig. 27. The points on the upper curve are obtained by the method previously described and are uniformly about four times as large as those of the lower curve. Apparently the methods

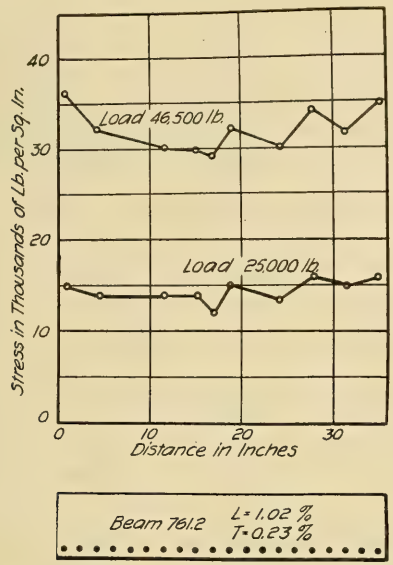


FIG. 21.—STRESS DISTRIBUTION DIAGRAM FOR 36-IN. BEAM WITH TRANSVERSE REINFORCEMENT.

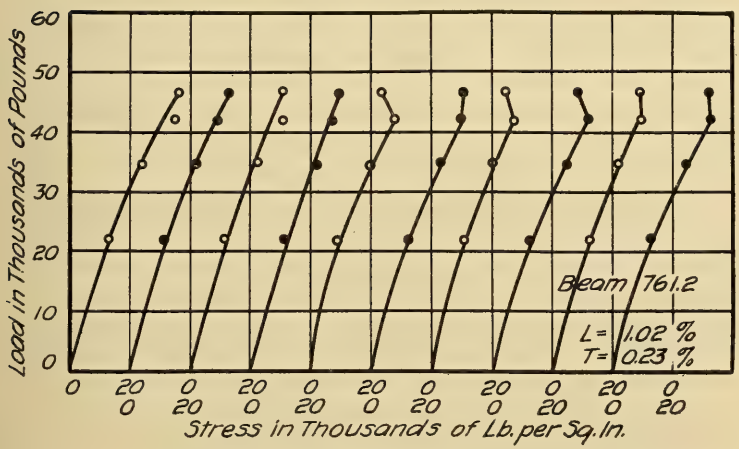


FIG. 22.—LOAD-STRESS DIAGRAM FOR 36-IN. BEAM WITH TRANSVERSE REINFORCEMENT.

which give fairly consistent results for the beams having no transverse reinforcement are not applicable where transverse steel is present. Probably when the concrete fails in tension along a longitudinal section the stiffness of the test piece as a transverse beam becomes much less, and a much larger part of the reaction is concentrated near the center of the width of the beam. This

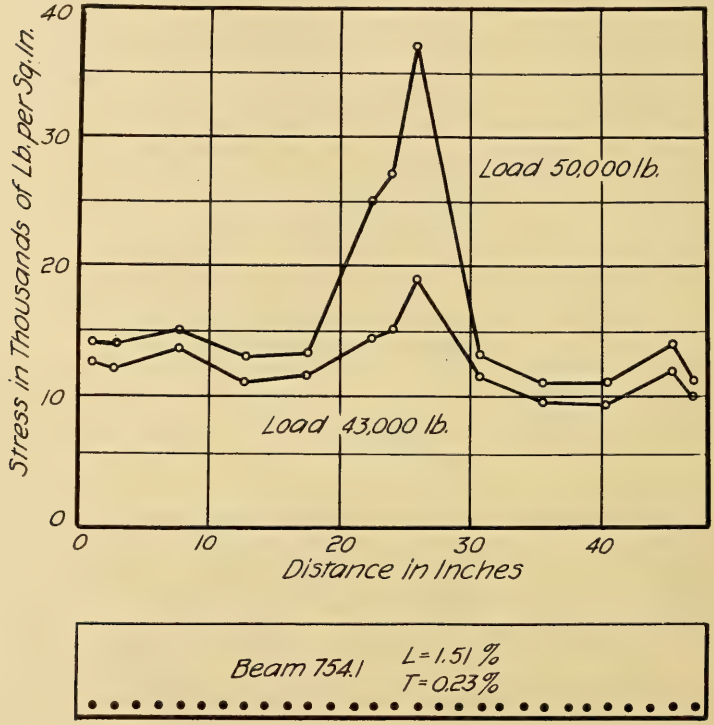


FIG. 23.—STRESS DISTRIBUTION DIAGRAM FOR 48-IN. BEAM WITH TRANSVERSE REINFORCEMENT.

would make the transverse moment much smaller in proportion to the load on the beam than for beams with no transverse reinforcement. There are other phenomena which apparently have a bearing upon this matter but which are somewhat conflicting and too complicated to be discussed in a short space.

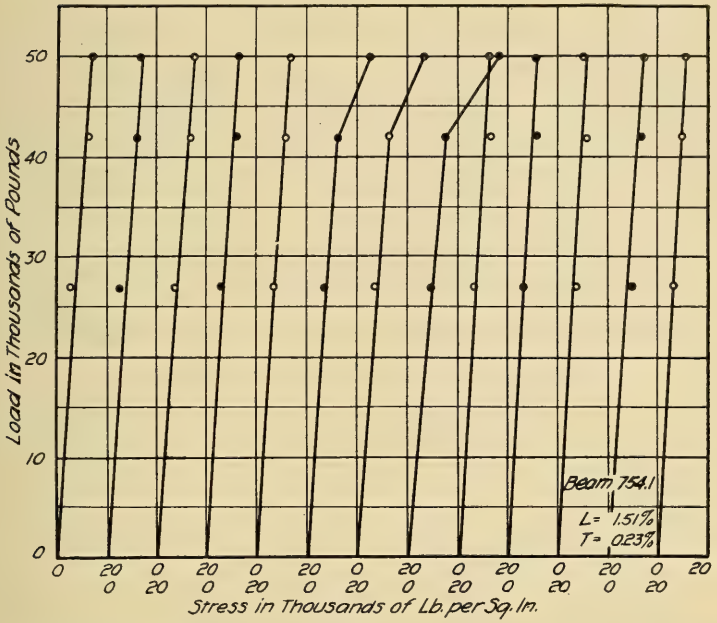


FIG. 24.—LOAD-STRESS DIAGRAM FOR 48-IN. BEAM WITH TRANSVERSE REINFORCEMENT.

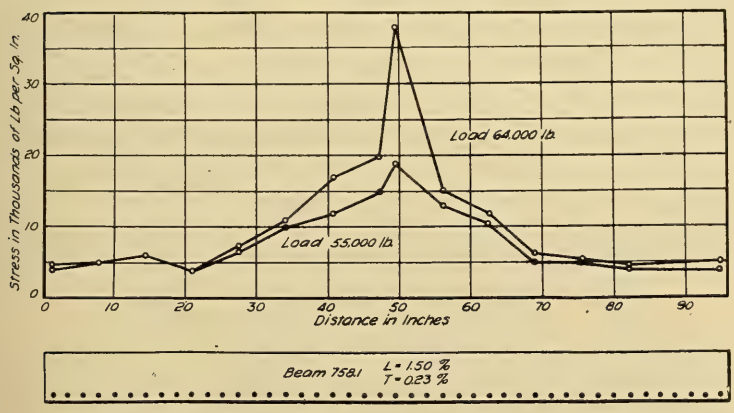


FIG. 25.—STRESS DISTRIBUTION DIAGRAM FOR 96-IN. BEAM WITH TRANSVERSE REINFORCEMENT.

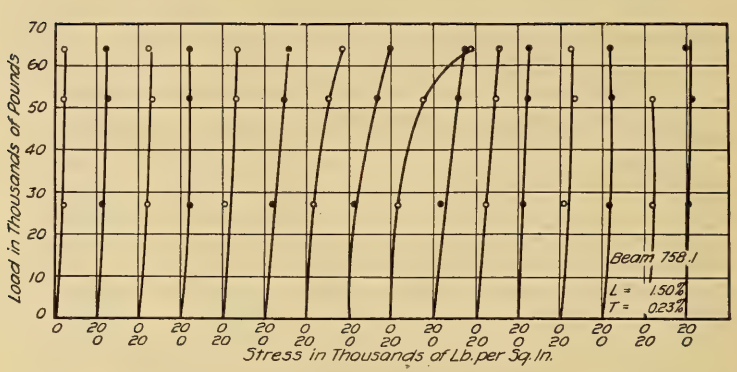


FIG. 26.—LOAD-STRESS DIAGRAM FOR 96-IN. BEAM WITH TRANSVERSE REINFORCEMENT.

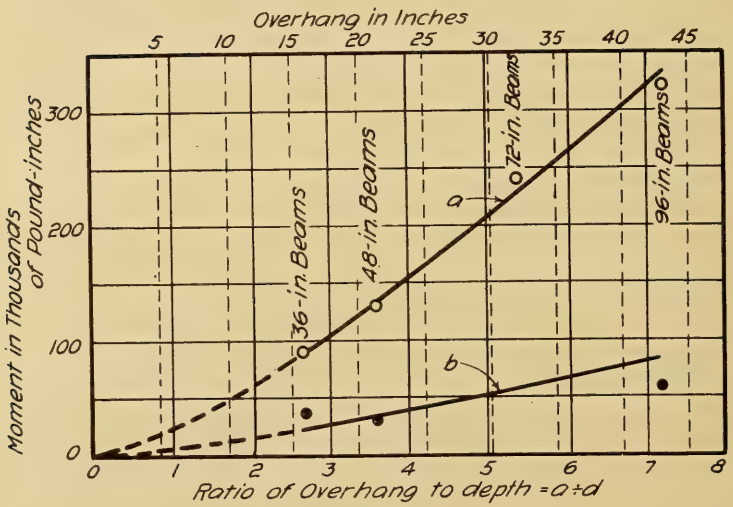


FIG. 27.—TRANSVERSE MOMENTS IN BEAMS HAVING TRANSVERSE REINFORCEMENT.

Note: Curve *a* assumes load on transverse beam proportional to stress in longitudinal steel. Curve *b* is summation of products of transverse steel stresses and arm of resisting couple.

Effect of Variation in Amount of Longitudinal Reinforcement.—To show the effect of a variation in the percentage of longitudinal reinforcement when the percentage of transverse reinforcement is kept constant, Fig. 28 was prepared from the average measured stresses in beams having 1.0 and 1.5 per cent respectively of longitudinal reinforcement. In this figure the stresses for individual beams were plotted. A mean stress distribution curve was then drawn for the beams having 1.0 per cent and one for the beams

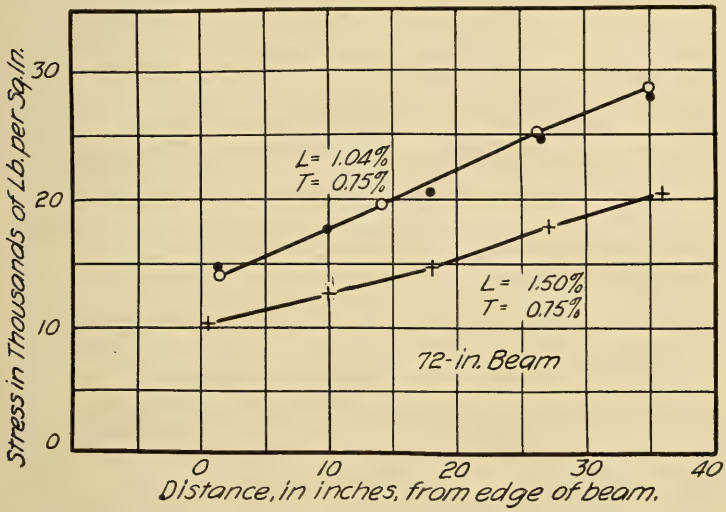


FIG. 28.—DIAGRAM SHOWING EFFECT OF VARIATION IN LONGITUDINAL REINFORCEMENT.

Legend { + Stresses in 1.50% beams. ○ Stresses in 1.04% beams. • 1.50/1.04 x stresses in 1.04% beams.

having 1.5 per cent of longitudinal reinforcement. Multiplying the stresses shown in the 1.5 per cent curve by the ratios of the two percentages $\left(\frac{1.5}{1.0}\right)$ the resulting values agree closely with those of the 1.0 per cent curve. This relation is, of course, to be expected with narrow beams. Its significance here is that variation in longitudinal reinforcement affects only the unit stress at any part of the beam but does not affect the total stress at that point. In other words, the distribution of stresses seems not to be affected by variation in amount of longitudinal reinforcement.

Effect of Variation in Amount of Transverse Reinforcement.—The effect of increasing the amount of the transverse reinforcement while the amount of longitudinal reinforcement is kept constant is towards making the stresses in the longitudinal rein-

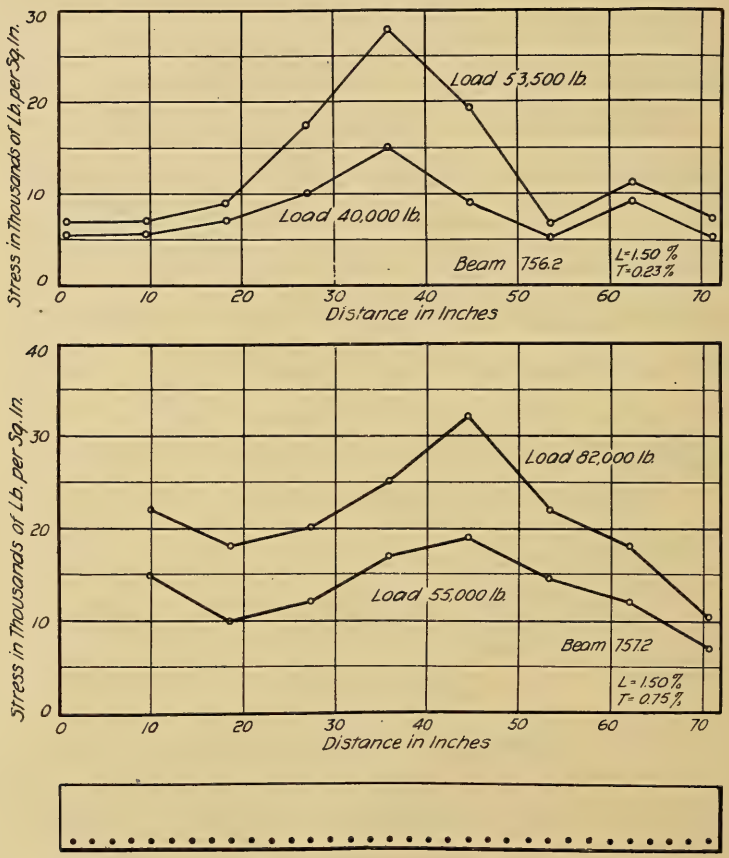


FIG. 29.—STRESS DISTRIBUTION DIAGRAMS SHOWING EFFECT OF VARIATION ON AMOUNT OF TRANSVERSE REINFORCEMENT.

forcement equal throughout the width of the beam at a given load. This is shown in Fig 29 by a comparison of the upper curve for beam 756.2 (load 53,500 lb.) with the lower curve for beam 757.2 (load 55,000 lb.). No expression for relation between the amount

of transverse reinforcement and its effect in equalizing the stress in the longitudinal steel has been determined.

Stress in Transverse Reinforcement.—In Fig. 30 the stress at various positions on a transverse reinforcing bar are plotted as ordinates and distance from the center of the beam as abscissas.

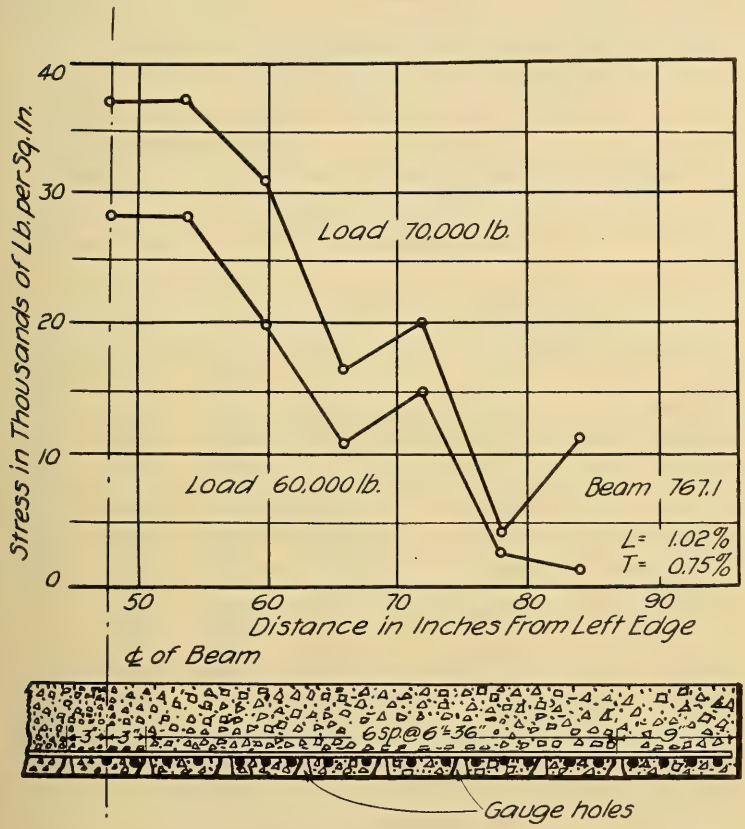


FIG. 30.—DIAGRAM SHOWING STRESSES IN TRANSVERSE BAR.

This curve indicates that the stress reaches a maximum at about the edge of the loading block, as was assumed in the calculation of the modulus of rupture for the beams having no transverse reinforcement. It was thought that by means of the stress curves for transverse rods and for the concrete above it in a transverse

direction the moment curves and hence the distribution of the load producing the transverse stresses might be obtained, but there are indications of such complications in the action as not to permit of so simple a solution.

Deformations in Concrete.—In a few of the tests deformations in the concrete were measured in both the longitudinal and the transverse directions. The stress distribution in the longitudinal direction appears to be much the same as for the reinforcement.

TABLE II.—COMPARISON OF CALCULATED STRESS IN LONGITUDINAL REINFORCEMENT WITH AVERAGE OF THE MEASURED STRESSES.

Beam No.	Width, in.	Per cent Reinforcement.		Load Considered, lb.	Measured Stress, lb. per sq. in.		Calculated Stress, lb. per sq. in.
		Longitudinal.	Transverse.		Maximum.	Average.	
750.1	24	1.07	0.00	12,000	38,500	34,500	48,900
751.1	24	1.07	0.00	37,000	35,000	29,600	37,700
752.1	24	1.56	0.00	41,500	37,000	33,600	29,800
762.1	36	1.02	0.00	38,500	26,500	25,300	27,300
763.1	48	1.04	0.00	31,600	42,000	15,630	16,500
763.2	48	1.09	0.00	40,200	42,000	21,400	20,200
753.1	48	1.51	0.00	42,500	21,000	12,300	15,900
753.2	48	1.51	0.00	42,000	22,500	15,570	15,560
765.1	72	1.04	0.00	32,000	20,000	10,300	11,100
765.2	72	1.04	0.00	40,500	29,000	10,960	14,100
761.1	36	1.02	0.23	38,600	34,000	30,100	27,500
761.2	36	1.02	0.23	46,500	35,000	33,000	33,100
754.1	48	1.51	0.23	50,000	37,000	16,230	18,530
754.2	48	1.51	0.23	57,000	36,500	21,300	21,100
764.1	48	1.04	0.78	66,000	42,000	35,800	34,300
764.2	48	1.04	0.75	65,500	38,000	36,200	34,100
756.1	72	1.50	0.23	67,000	29,000	13,620	16,600
756.2	72	1.50	0.23	53,500	27,700	12,920	13,260
766.1	72	1.04	0.75	80,700	42,000	25,000	28,000
766.2	72	1.04	0.75	61,500	33,000	26,000	21,300
757.1	72	1.50	0.78	76,500	31,000	19,700	18,950
757.2	72	1.50	0.75	82,000	32,000	21,500	20,300
758.1	96	1.50	0.23	64,000	38,000	9,800	11,920
758.2	96	1.50	0.23	59,500	42,000	10,800	11,080
767.2	96	1.02	0.75	73,800	39,500	24,000	19,700
767.1	96	1.02	0.75	71,500	37,000	17,500	19,090

In the transverse direction the deformations were very small, this being due probably to the large amount of concrete available to take the transverse compression. There was some indication that the stress passed from compression near the edge of the loading blocks to tension nearer the edge of the beam. In all cases where this phenomenon was observed the amount of tension was so very small that more information is needed to establish the fact of its existence. Its meaning, if present, must be that not all transverse strips act as beams and that if the beam as a whole

acts as a transverse beam, other portions than the one under consideration must take an increased amount of compression due to the fact that tension in concrete is present on the strips which pass near the loading block.

Average Measured Reinforcement Stresses.—The use of the ordinary formula $M_L = pfjbd^2$ for calculation of longitudinal steel stresses in wide beams gives results usually much lower than those which exist near the center of the width of the beam. However, it has been found that the average of the measured stresses is in

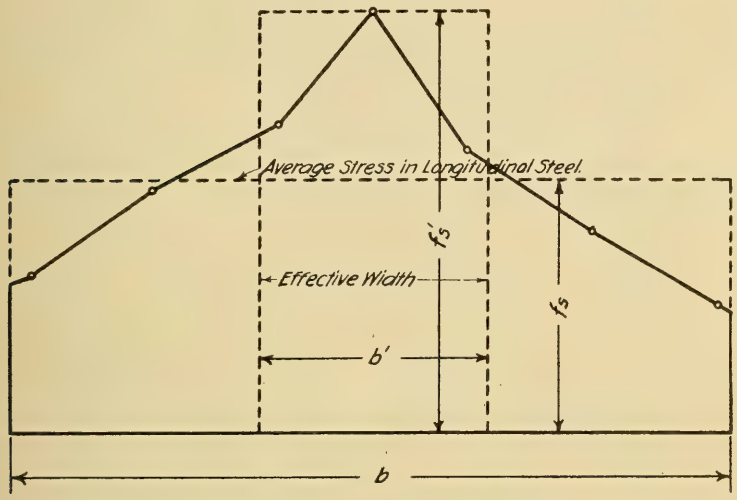


FIG. 31.—DIAGRAM SHOWING RELATION BETWEEN EFFICIENCY AND EFFECTIVE WIDTH.

reasonably close agreement with the stresses calculated for the same load. This is shown in Table II, in which the concrete is assumed to be stressed to 0.5 of its ultimate strength.

Relation between Efficiency and Effective Width.—The total stress in the longitudinal reinforcement is equal to the area times the average stress. But the average stress apparently is approximately equal to the stress calculated by the ordinary methods used for reinforced concrete beams. The total stress for any load can then be calculated. The significance of this fact is shown in the following paragraph.

In Fig. 31 the following notation is used:

f_s = the average stress in the longitudinal steel at the center of the span.

f'_s = the maximum stress in the longitudinal steel.

$\frac{f_s}{f'_s}$ = the efficiency of the beam.

b = the width of the beam.

b' = the effective width of the beam

Since the maximum stress will govern the design, f'_s will be the designing stress. By effective width is meant the width in which if the stress in all the longitudinal reinforcement be considered to be equal to the maximum, the total stress will be equal to the actual total stress in the beam.

From the above relations it will be seen that the reinforcement stress on a section at the center of the span is represented to some scale by bf_s and to the same scale by $b'f'_s$ provided that the longitudinal steel is uniformly spaced.

Therefore $$bf_s = b'f'$$

$$\frac{b'}{b} = \frac{f_s}{f'_s}$$

and

$$b' = b\frac{f_s}{f'_s}$$

From the data obtained in the tests values of the ratio $\frac{f_s}{f'_s}$ (here termed efficiency) have been calculated and are given in Fig. 32.

The efficiency generally decreases as the load increases and these values are usually those for the maximum load on the beam. Therefore, at the stage when the reinforcement is stressed only to an ordinary working stress the efficiency and the effective width would be larger than is indicated in the diagram. However, it seems that the efficiency at the maximum load is the one to be used in design since the real factor of safety must be based on the ultimate strength of the beam.

Relation between Rigidity of Supports and Stress Distribution.—Such a distribution of reactions as is shown in Fig. 9 along the beam f would be expected to result in a considerable uniformity

of the longitudinal stresses at the center of the span and a consequently high efficiency. However, it will result also in high stresses in the transverse reinforcement and a consequent tendency toward slab failure. The distribution of reaction shown in Fig. 10 will produce smaller transverse stresses but will tend to cause

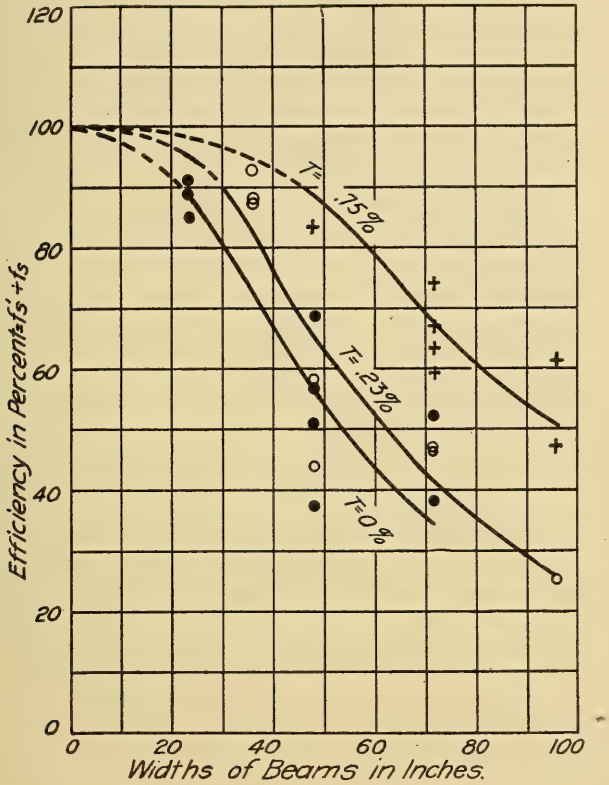


FIG. 32.—DIAGRAM SHOWING RELATION BETWEEN EFFICIENCY AND WIDTH OF BEAM FOR VARIOUS AMOUNTS OF TRANSVERSE REINFORCEMENT.

greater stresses in the central longitudinal elements and hence will result in smaller efficiencies. For economical use of material then it would seem that a balance must be found between too great rigidity of support and no transverse reinforcement, and too great flexibility of support and the large amount of transverse

reinforcement which would be required. No attempt is made here to show how this balanced condition is to be found. The stress distribution curve for beam 762.2, Fig. 13, illustrates what might be expected to result from too great flexibility of supports with too little transverse reinforcement, although it is not known that in this case the result shown was brought about in this way.

The tests which so far have been made have not yielded much data that are directly applicable to the design of beams of such dimensions as would be used in practice, but the determinations of effective widths which have been made may be expected to be of some use as a guide. Some data have been obtained on the variation in effective width due to variation in the depth and in the span of beams but the tests do not cover sufficient range and were not made on a sufficient number of test pieces to afford a basis for conclusions. The method used for calculating transverse bending moment gives reasonable results for the beams having no transverse reinforcement, but its use depends upon a knowledge of the distribution of the longitudinal stresses. Since an expression showing this distribution has not been determined, this method could not be applied to design.

The indications that in wide beams the effective width is not changed by variation in the percentage of longitudinal reinforcement and that the average of the stresses in the longitudinal reinforcement are in close agreement with the stresses calculated by ordinary methods, are of importance in design. Most of the information gained even if not of great practical use indicates progress towards a solution of the problems presented in the design of wide beams. Further tests are projected which should give valuable information.

DISCUSSION.

Mr. W. A. Slater.—Initial stress is a matter that would have to be taken into consideration in interpreting the results of the tests if conclusions were based on the loads carried by the beams. No doubt initial stress in the steel, if present, would affect the strength of a beam. However, in these tests measurements were taken of the stresses developed in addition to whatever initial stresses were there, and deductions are based to a greater extent upon the observed stresses than upon the load carried. As far as known, the stresses developed within the elastic limit are proportional to the load applied, whether initial stresses are present or not. Therefore, the conclusions as to stress distribution would not be affected by disregarding the initial stresses. This is equivalent to choosing the zero line for stress (in the load-stress curve) at a point somewhat above the actual zero line, hence we might find that failure came at a lower stress than we should expect. This principle of proportionality between the applied load and the stress developed thereby was assumed by Mr. Lord in the report of the Larkin Building test, but a different use was made of it. In his test the initial (dead load) stress was important and he used this principle to determine the amount of it; in the tests of wide beams I have used the principle as justification for disregarding the effect of initial stresses on the stress distribution. He measured stresses from an assumed zero line, at which an unknown stress was present due to the dead load. Assuming the change in load to be proportional to the change in stress, the stress due to the dead load was determined. I had not given much thought to the matter of initial stress in this particular connection because it had not occurred to me as having a bearing. Mr. Slater.

Mr. Arthur R. Lord.—I would like to ask Mr. Thompson how he figures stresses around the right angle in a rigid frame? This is a matter that has always troubled me in design. Mr. Lord.

Mr. Sanford E. Thompson.—In steel design, this is a difficult thing to do. In reinforced concrete design, the problem is simpler, because the concrete at the corners can be shaped in such Mr. Thompson.

Mr. Thompson. a way as to give the required thickness at any point, and can be reinforced both for tension and compression.

Mr. Lord. MR. LORD.—The stress around that right angle theoretically would be very large indeed, very near to infinity, and I was wondering how much you actually consider it in designing structures.

Mr. Thompson MR. THOMPSON.—Each case has to be considered by itself, so as to adjust the design to fit conditions. By thickening the concrete so as to form an angular, or, better still, a curved haunch, the requisite thickness of concrete can be obtained and the steel for tension and also compression can be inserted either on a curve or with slight bends, so that the stresses can be figured with fair accuracy at any point. The bond of the concrete to steel permits it to take stress even at points where it does not run directly parallel to the line of stress.

When preparing the paper, it was the original intention to show illustrations of a large number of designs that have been actually constructed abroad, where the rigid type of frame has been employed. Lack of space, however, prevented the introduction of this into the paper.

Mr. Lindau. MR. A. E. LINDAU.—Mr. President, did I understand Mr. Lord to assume that the bars would be bent at a right angle, or was that merely the graphical representation of what such a structure might be, with considerable variation of detail?

Mr. Lord. MR. LORD.—I did, on the ground simply that in a curved beam your stress increases very much more rapidly, and when you get to the end and compute stresses around the right angle you get exceedingly high stresses.

Mr. Lindau. MR. LINDAU.—If we make a design to cover such a case, the detail will have to be taken care of very much as it is in the region of the wall, when you change the direction of the back of the wall, running the piers past, or make a full long sweep so you can bend the bars over a very long radius to avoid sharp corners. What the stresses might be in either of the two cases, where you have very large loads or where you load the bars, is problematical.

Mr. Condron. MR. T. L. CONDRON.—I would like to ask Mr Lord if there were any vertical bars running from the floor side to the span of the girder.

Mr. Lord. MR. LORD.—The wall section varied a great deal. In some places underneath the wall panel there were shipping doors in the

first story and in some places there were windows. In the particular panel on which was placed the final heavy loads there was a window, so that there was no concrete below the second floor slab; above this slab there was a joint in the concrete due to the stopping of it at the floor level, the spandrel wall above the floor level being cast with the third floor slab. The bars did not run into the floor slabs and they were not anchored. Mr. Lord.

THE PRESENT STATUS OF UNIT METHODS OF REINFORCED CONCRETE CONSTRUCTION.

By John E. Conzelman.*

Unit construction has arrived. The large number of structures built after this method during the last few years shows that it can be successfully used for practically all kinds of reinforced concrete construction. We now know that contracting companies exploiting unit methods have a fair chance of obtaining contracts in open competition; in fact, it can be truthfully said that where conditions are favorable, unit methods have the old forms of construction hopelessly beaten.

In a great many types of buildings unit construction is so obviously the logical method that, to one familiar with this method, the usual construction seems most crude and cumbersome.

The change in the attitude of architects and engineers toward this type of construction is nothing short of marvelous. It is very gratifying to those of us who, during the last few years, have advocated unit construction in the face of almost universal adverse criticism, to be able to say that our labors have not been in vain, and that the present status of the unit methods of reinforced concrete construction is very different from that when Mr. Charles D. Watson read before this Association in 1908, his paper on "Factory Made Concrete."

The general subject of unit construction will be very briefly considered before taking up the subject of this paper. The term "monolithic concrete" is used to indicate concrete that has been deposited into forms in the usual manner. This word "monolithic" has been responsible for some misapprehension as to the nature of reinforced concrete structures. No building or large structure is monolithic, using the word in its proper meaning. We now better realize what we have always known, viz., that the usual reinforced concrete building is not "a monolith carved out of the solid rock" as we used to read in some of the catalogues,

* Vice-President and Chief Engineer, Unit Construction Company, St. Louis, Mo.

but has a great many cracks and joints. The cracks are a necessary result of the method of construction which involves the pouring of a large expanse of concrete at one time, which must shrink as it dries out, and cracks inevitably follow. Temperature stresses are also a prolific source of cracks. Joints occur whenever the work is stopped, even though the interruption in the pouring is of very short duration.

Admitting then the existence of joints and cracks, does it not appear logical to adopt a method of construction which eliminates objectionable shrinkage, partially eliminates temperature stresses, and finally makes it possible to locate all joints just where the designer intended they should come?

The opportunity for properly inspecting the finished product before it is incorporated into the structure is a great advantage peculiar to unit construction. You do not as a rule (at least, if you are at all particular as to quality) buy finished lumber before the bark is taken off of the tree, and yet a great many people are satisfied to purchase reinforced concrete just that way.

You cannot place too high a value on this one great advantage of unit methods. Just think of the loss of life and property that would have been averted if the art of reinforced concrete construction had begun with unit methods. No accidents from premature removal of forms can occur when unit methods are used. The subject of unit construction has been discussed to such an extent recently that further exposition of its advantages seems superfluous. However, as the subject is a most important one, mention will be made of a few of the more obvious points of superiority.

Under unit methods it is entirely possible to follow a well-thought-out program for the work in hand. The forms are bedded on the ground in a suitable casting yard and the work so arranged that a routine operation results; this means that it has finally been possible to enforce factory methods in construction operations, and factory methods applied to the contracting field mean order out of chaos. Forms for similar units are grouped together, thus making inspection and the checking of the reinforcement an easy matter. The placing of the steel can be done more easily and with greater accuracy; and the reinforcement is not so likely to be displaced as in a monolithic building where there is neces-

sarily considerable walking or wheeling over the steel that has been placed. It is possible to make very thin sections when units, such as wall slabs, are cast flat; this results in economy of materials. Unit forms can be kept clean; shavings, sticks, etc., cannot be washed into them from connecting forms, as is the case in monolithic work.

The question of inspection and testing of the units has been treated fully above. It may, however, be desirable to explain how shrinkage is eliminated. While each individual unit will have its own shrinkage stresses, these are not cumulative and do not affect the structure as a whole; the shrinkage of the small amount of grout used cannot affect the structure.

It would be possible to give many other advantages of unit construction methods, but the speaker will close by saying that in his contracting experience unit methods have made possible the adoption of reinforced concrete for buildings that would otherwise have been built of ordinary construction on account of the excessive cost of monolithic construction.

Although there have been several papers on the subject of unit construction read before this Association, it will be necessary to briefly review the development of unit methods in order to properly set forth present accomplishments.

During 1908 Mr. William H. Mason, superintendent of the Edison Portland Cement Company, New Village, N. J., built two kiln buildings, a clinker storage building and portions of a cement storage building, using unit columns, girders and roof slabs. No attempt was made to connect these units by means of projecting bars and grout, but the columns and girders were fastened together by means of steel splice plates and belts. A complete description of this work will be found in the 1908 Proceedings of this Association.* In the same Proceedings appears a paper on "Factory Made Concrete," by Mr. Charles D. Watson,* which shows that the author was keenly alive to the importance of unit methods, and while the advantages of unit methods under factory conditions of manufacture are brought out, the examples cited were mostly applications of the precast floor joist. In the 1910 Proceedings there is also a paper on unit construction by Mr.

* See Proceedings, Vol. IV, 1908, pp. 49 and 97.

Watson,* in which he cites quite a few buildings constructed by this method, which indicates that considerable progress had been made since his first paper. Reference is therein made to the following buildings:

A building constructed according to the Visintini method, under the direction of the Concrete Steel Company, at Reading, Pa., for the Textile Machine Works. The cost of this building was reported excessive, and there is no record of other buildings constructed in this country by this system.

A car barn for the Central Pennsylvania Traction Company, built at Harrisburg under the direction of Mr. Mason D. Pratt, is noteworthy in that the walls as well as the columns, girders and roof slabs are of unit construction. This building is of similar construction to the kiln buildings erected by Mr. Mason, in that no attempt was made to obtain a rigid structure by specially designed joints between the members.

A building for the Onondaga Litholite Company, designed and constructed under the direction of Mr. Watson, is an interesting example of unit construction. This is a one-story building with bolted connections between members. Mr. Watson devotes a large portion of his paper to a description of different unit floor systems, and states that thus far the most extensive application of unit methods was in such construction. This fact is mentioned because it shows that only two years ago unit construction, as we now use that term, was not very highly developed, at least in the Eastern States.

Mr. Robert A. Cummings of Pittsburgh has been very successful in his application of unit methods to certain types of factory buildings.

Outside of building construction, unit methods have been extensively used by the railroad companies in the construction of bridges. Notable examples of pioneer work of this kind can be found in the track elevation work in Chicago, Ill. Several arch bridges of the three-hinge type have been constructed in the extreme west under patents granted to W. M. Thomas; one of these has arches of 103 feet span.

At the convention in Kansas City last year J. L. Darnell

* See Proceedings, Vol. VI, 1910, pp. 391.

read a paper on unit construction in which he reviewed and illustrated with the stereopticon the progress made.

The Ransome Engineering Company of New York has also developed a unit method, and has constructed quite a few buildings; this method is a great advance over the constructions described, as it can be applied to buildings of more than one story. In this system, however, the floor slab is poured "in situ," making the construction only partially unit.

As the floor slab concrete is a large percentage of the total, it seems to the speaker that a large part of the savings possible by unit methods is lost by this method. There have been quite a few buildings constructed by this method, of which the following are the more important:

The four-story building of the United Shoe Machinery Company at Beverly, Mass. This building is of pleasing architectural design, and is an addition to existing buildings which also have been built under Mr. Ransome's supervision. It is stated that quite a substantial saving in cost was effected by the adoption of the unit method.

During the last two years several buildings have been constructed in Boston by this method.

The first application of this method was in the construction of an office building, 50 x 60 ft., two stories and basement, with brick walls, for the Foster-Armstrong Company, near Rochester, N. Y.

As the Ransome system is important in the development of unit methods, and has been demonstrated on considerable work, it will be more fully described.

In this system the columns, girders, beams and wall slabs are of unit construction; the column bars are not continuous but the concrete section is enlarged at the top and bottom so that it can transmit the load without assistance from the steel. The ends of the girders are enlarged so as to cover the lower column head and provide a base for the upper column. After the girders are erected the beams are set into notches in the girders. The beams are provided with ledges for supporting the centering for the slab. The columns have a central hole for grouting; reinforcing bars which lap each other are placed in this hole, forming a steel tie through the columns. Abutting beams are

tied with a steel crimp after the practice in masonry construction, and the girders are similarly connected. It will be seen, therefore, that in this construction the stability of the structure is to an extent, at least, dependent upon the unifying action of the monolithic slab. The unit girders and beams are proportioned to carry the dead load during construction; after the slab has hardened the strength of the beams and girders is largely increased. This is due to the fact that if sufficient steel extends into the

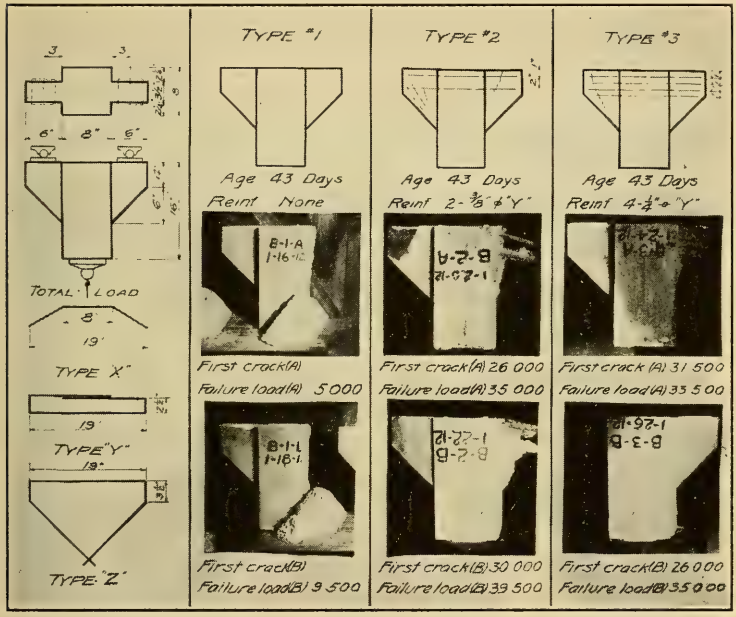


FIG. 1.—DIMENSIONS AND TESTS OF BRACKETS.

slab, the slab and beams act together as when they have been poured at the same time.

A series of tests made at Harvard University, under the direction of Mr. Ransome, demonstrated that the beams and girders became T-beams after the slab had hardened; details of these tests will be found in the book published by Ransome and Saubrey entitled, "Reinforced Concrete Buildings." In passing it may be said that experiments made by the speaker. some of which will be described later, corroborate this conclusion,

Even in this well-devised method there is still no attempt made to obtain the rigidity and continuous action that should be the distinguishing feature of either unit or monolithic reinforced concrete construction. A successful unit system must therefore be one which can be applied to the construction of practically all structures which would ordinarily be built of reinforced concrete. An example will illustrate the point the speaker is trying to make: Suppose it was desired to construct a very tall building on a very narrow lot, in which case it would

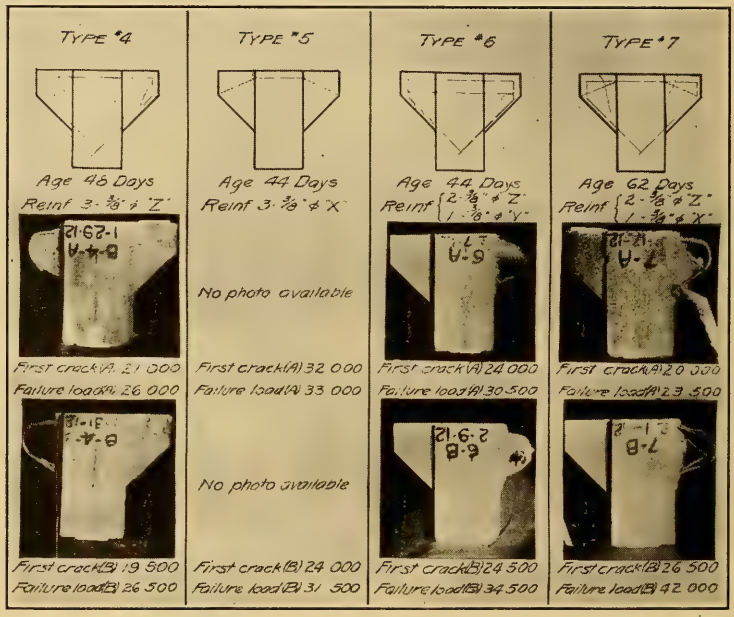


FIG. 2.—DIMENSIONS AND TESTS OF BRACKETS.

be necessary to consider the stresses due to wind loads; it is evident that a unit method of construction could not be used unless the joints were so designed as to secure the necessary stiffness and knee brace action.

The Unit Construction Company of St. Louis has such a system, and during the last five years has so perfected its methods that structures which have all the rigidity characteristics of the monolithic type can be erected in which only three per cent of

the total concrete is poured "in situ." All the joints in this system are made by means of projecting reinforcement; the bars projecting from abutting units are made to lap sufficiently to develop the requisite strength when imbedded in the grout which is used to make the connection. As this company is by far the largest factor in the unit construction field (in fact, the only large company exploiting unit methods), a description of the present status of unit construction will necessarily treat largely of their work.

Unit construction affords a great field for research, and in order to develop and demonstrate the methods, a testing laboratory was equipped with a 50,000-lb. testing machine and other

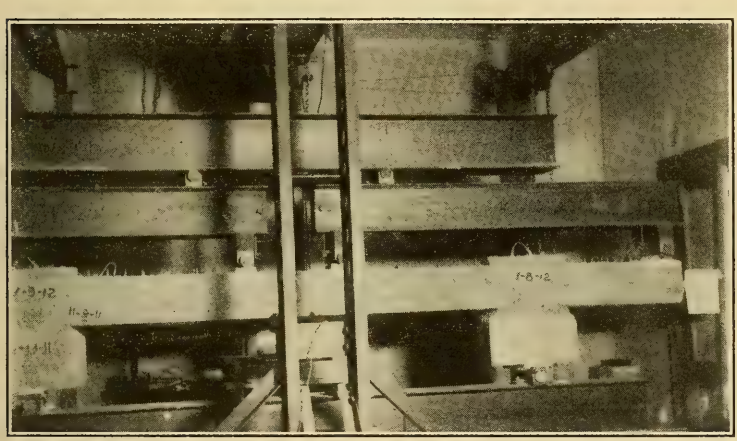


FIG. 3.—TEST OF CONTINUOUS GIRDER.

necessary apparatus. This laboratory has proven to be a very valuable adjunct to the engineering department. Among the experiments that have been made are two series of sufficient interest to merit description: One was devised to throw some light on the subject of bracket design; the other to show the action of girders with grouted connections. Tests on brackets were made because there is no satisfactory analysis for their design and no experimental data is available. It is hoped that the information derived from these experiments may be of value to the profession.

The dimensions of the test brackets are shown in Fig. 1;

the concrete was a 1 : 2 : 4 mixture of Merrimec River sand and gravel. They were tested in an inverted position on account of greater ease in handling; the load was applied through a hardened steel ball bearing in a cup-shaped depression in a heavy steel plate. The brackets rested on 1 x 4-in. steel plates carried on rollers as indicated in Fig. 1. Plaster of Paris joints assured an even bearing between plates and concrete. Seven types were tested; one without reinforcement, and six with reinforcement as shown in Figs. 1 and 2; three types of reinforcement were used, singly, and in combination. Type X was a short length of bar bent downwards at each end; type Y a single bar bent in the shape of a rectangle; type Z was a single bar bent to follow the shape of the bracket; type Z reinforcement is the type usually employed. Two brackets of each type except type No. 7, which were tested at the age of 62 days, were tested as near as possible at the age of six weeks; the photographs show clearly the con-

FIG. 4.—WING GIRDER AFTER TEST.

dition of the brackets after failure; the load causing the first noticeable crack, and the load causing failure and the position and extent of the cracks are also shown (Figs. 1 and 2). As was to be expected none failed in tension. The failures were due to shear or diagonal tension, and insufficient bond. The action is similar to that which exists at the end of a beam; the condition is aggravated, however, by the difficulty of securing sufficient bond to develop high stresses in the reinforcement.

Although the conclusions derived from these tests are not final, yet the fact is brought out that the type of bracket ordinarily used is the least efficient of those tested. The failure is fairly sudden and the concrete is almost completely destroyed. The most satisfactory is type Y, of the closed loop; with a number of loops spaced close together, the failure is gradual, and even after the concrete is partially destroyed, considerable load-

carrying capacity remains. This feature is shown by type 3, Fig. 1. The average shear per square inch in the vertical plane of junction with the column at first load and at failure is given in the following table:

TYPE.	SHEAR AT FIRST CRACK, LB. PER SQ. IN.	SHEAR AT FAILURE, LB. PER SQ. IN.
1	100	100
2	400	530
3	410	490
4	285	375
5	400	460
6	345	465
7	330	470

A study of these tests leads to the conclusion that the most efficient reinforcement is the Y type loop bars, some of which are bent downwards into the bracket. Another series of tests to determine this point is now being made.

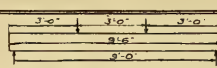
As an understanding of the details of this system will assist in understanding the purpose of the girder tests, a brief outline of the methods will be given. The girders rest on column brackets; the arrangement of the main reinforcement for the girders follows usual practice, some of the bars being bent up and carried over the support; these bent-up bars project from the girder and lap the bars from the abutting girder; the length of lap is made sufficient to develop the requisite stresses in them. After the grout is poured the girders are as truly continuous as in ordinary monolithic construction. The stirrups project above the girder and form loops so as to make a strong connection with the concrete on top of the girder. The floor slabs rest for their full width on the girder ledge, projecting bars extending into the grout space over the girder. It is evident that after the grout has hardened the strength of the girder will be greatly increased on the compression side. In other words, T-beam action results and the experiments were intended to throw light on this action.

A continuous girder test is shown in Fig. 3. The concrete blocks representing the column caps are supported on rollers, the cantilever ends and interior girder are grouted to the column cap in the usual way. Fig. 4 shows a wing girder after test. Note that the wings follow the curve of deflection.

The results of the tests on simple and wing girders are given in the Table I. The wing girders failed in tension and the simple girders in compression. The concrete was a 1 : 2 : 4 sand and gravel mixture; the reinforcement high elastic limit corrugated bars. The load was applied at the third points; the extension of the steel was measured in a distance of 8 in. at mid-span; the deformation of the concrete was also measured, steel buttons

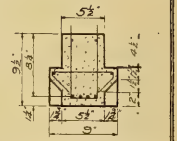
TABLE I.—RESULTS OF TESTS OF SIMPLE AND WING GIRDERS.

Stress in Steel-lbs per sq in as measured by extensometer
Load is total applied load

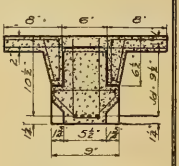


Concrete Deformation given in ten thousandths of one inch as measured by extensometer in length of 8 inches

GIRDER	LOAD IN POUNDS	MAXIMUM DEFLECTION IN INCHES	STRESS IN STEEL	CONCRETE DEFORMATION LEFT WING	CONCRETE DEFORMATION MIDDLE	CONCRETE DEFORMATION RIGHT WING
G-1-A	8 000	.175	23 400		66	
	16 800	.550	56 100		115	
	17 000					
G-1-B	8 000	.190	23 800		66	
	16 000	.550	58 600		239	
	16 300					
G-1-C	8 000	.170	23 250		57	
G-1-D	8 000	.190	21 750		52	
G-2-A	7 600	.080	10 150	13	14	14
	16 300	.200	26 500	38	38	32
	26 600	.400	48 550	76	70	54
	30 000					
G-2-B	8 000	.075	12 850	19	18	16
	16 000	.185	23 200	41	42	39
	27 900	.380	58 150	89	88	88
G-2-C	7 900	.050	10 850	14	15	14
Made from G-1-C	16 000	.135	25 850	28	31	28
	27 300	.270	50 600	54	53	53
G-2-D	8 000	.055	11 750	15	17	14
Made from G-1-D	16 000	.130	25 850	31	33	30
	28 000	.265	50 050	59	63	60



SIMPLE GIRDER
STEEL { 2-½" ø Str
2-½" ø Bent



WING GIRDER
STEEL { 2-½" ø Str
2-½" ø Bent

being set for this purpose. Readings were made to the nearest one ten thousandth of an inch; corrections were made for temperature.

Referring to the wing girders, you will see that the deformation of the wings agrees closely with that of the concrete over the girder; this means that these wings carried their full share of stress and that full T-beam action was secured. The wing girders carried twice the load borne by the simple girders.

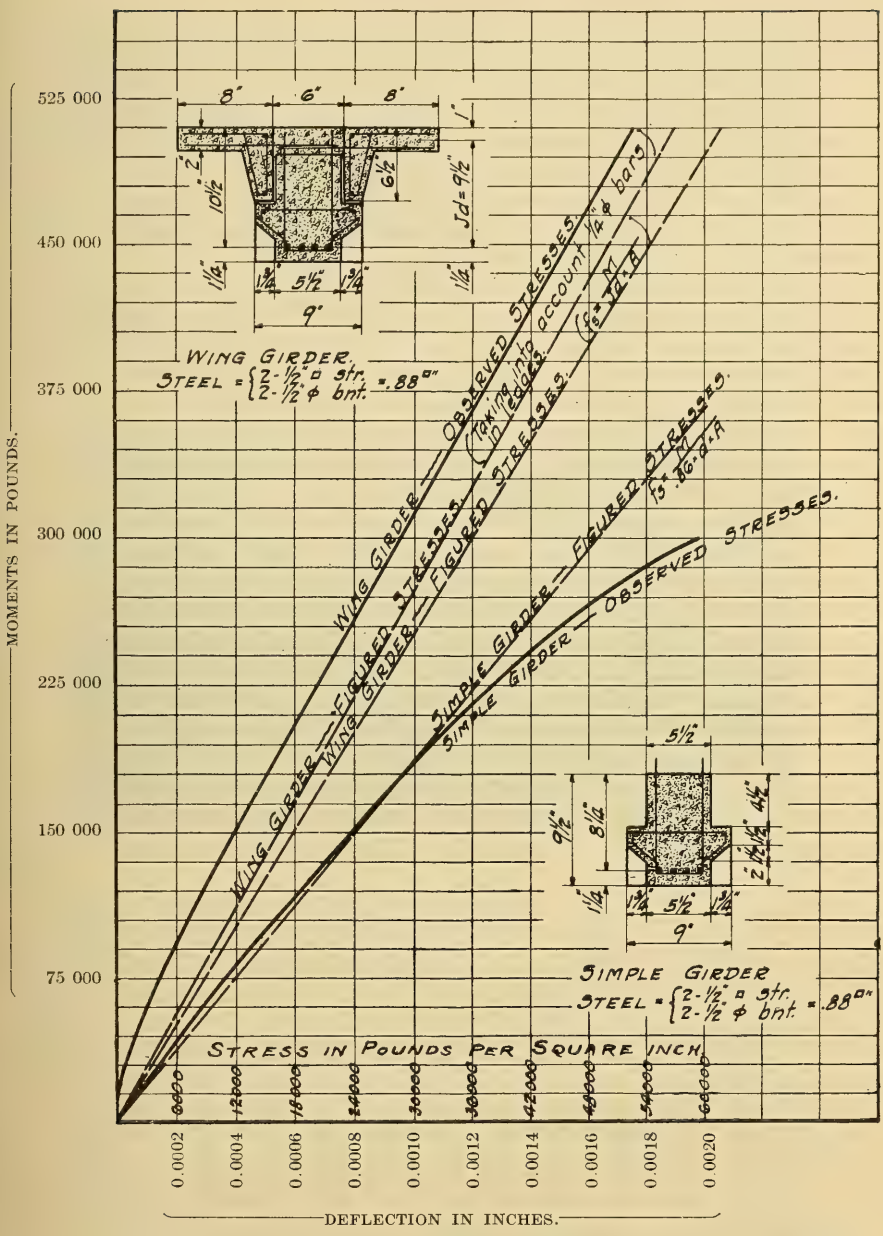


FIG. 5.—STRESS DIAGRAM (STEEL); AVERAGE OF ALL GIRDER TESTS. LOAD APPLIED AT THIRD POINTS. SPAN 9 FT. C TO C.

The diagram (Fig. 5) was prepared for the purpose of comparing the action of the wing and simple girders. A comparison of the observed stresses in the reinforcement with those figured by the usual formula is also given. The agreement between the observed and computed stress in the simple beams is remarkably close to 30,000 lb. in the reinforcement. Above this point divergence must be expected as the straight line formula used is not intended to apply to any but working loads.

Unit construction presents new problems to the contractor, and these cover not only the erection, but the casting of the units as well. The difficulties met with in pouring units differ from

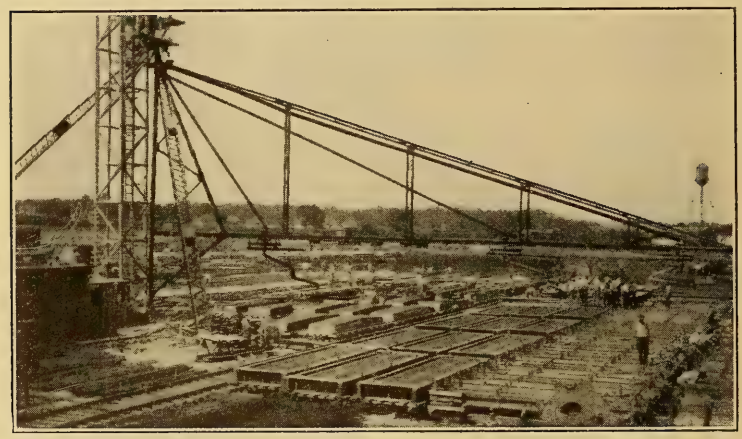


FIG. 6.—YARD LAYOUT, MEMPHIS, TENN.

those encountered in ordinary monolithic work. This results from the fact that in unit work it is necessary to fill a great many isolated forms each day, while in monolithic work a comparatively large amount of concrete can be poured in about the same location. In the attempt to meet these new conditions we have tried various types of mixing plants, two of which are of sufficient interest to merit description.

In the yard layout for the construction of a plant for the National Lead Company in St. Louis, the sand and gravel were received in hopper-bottom cars; a bucket elevator carried the aggregate from the hopper under the track to the bins over the mixer; this was mounted under the bins and arranged to dis-

charge into cars which were operated by a cable on an elevated track. These cars were stopped where required and the concrete dumped into the receiving hopper of the movable distributing spouts. As the spout hoppers were equipped with discharge gates, the flow of concrete could be readily controlled.

The casting yard was served by a stiff-leg derrick mounted on a tower. The derrick traveled the length of the yard and lifted the hardened units from the forms and placed them in the storage yard. A locomotive crane loaded the units onto flat cars and pushed them to the building site. The erection was done by a stiff-leg derrick mounted on a high traveler.

As far as the speaker has knowledge, the five-story building

FIG. 7.—COMPRESS BUILDING, MEMPHIS, TENN.

of this plant was the first building over one story in height built entirely of units including the walls.

Fig. 6 shows the yard layout for the Memphis Terminal Corporation work at Memphis, Tennessee. In this plant the casting yard is circular; the forms being arranged around the concrete distributor shown on the left. This distributor consists essentially of a structural steel arm, 130 ft. long, revolving through an angle of 270 deg. This arm is attached to a mast held by stiff legs anchored to heavy concrete footings. The arm carries three spouts as shown on the photograph and with the combination of motions obtainable, every point in the casting yard can be reached in a radius of 150 ft. The mixer and concrete hoist is

placed behind the mast, a three-way valve, operated by a man on top of the tower, directs the concrete into any one of the spouts.

The casting yard is served by two locomotive cranes that pick the units from the forms, load them on cars and push them to the buildings.

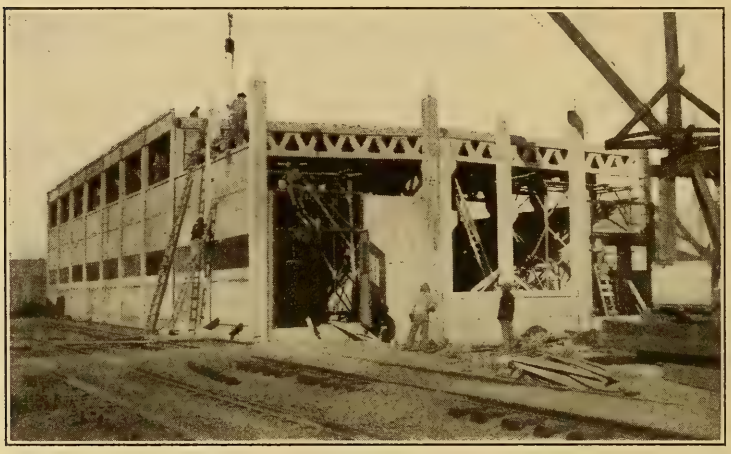


FIG. 8.—POWER HOUSE, POST, TEX.

Fig. 7 is an interior view of the compress building, 900 ft. long by 160 ft. wide. The actual time of erection being five weeks, or 30 ft. per day.

The construction of a cotton mill for the Postex Cotton Mills Company, at Post, Texas, is shown in Fig. 8; the unit lattice girders, columns and wall panels may be seen in position.

National Association of Cement Users.

HARRISON BUILDING,
PHILADELPHIA, PA.

STANDARD SPECIFICATIONS FOR PORTLAND CEMENT STUCCO

Paragraphs marked (*a*) apply only to back-plastered walls without sheathing.
Paragraphs marked (*b*) apply only to walls with sheathing.
All other paragraphs apply to both forms of construction.

Materials.

1. *Cement.*—The cement shall meet the requirements of the Standard Specifications for Portland Cement of the American Society for Testing Materials, and adopted by this Association (Standard No. 1).

2. *Fine Aggregate* shall consist of sand, crushed stone, or gravel screenings, graded from fine to coarse, passing when dry a screen having $\frac{1}{8}$ in. diameter holes, shall be preferably of silicious materials, clean, coarse, free from loam, vegetable or other deleterious matter.

3. *Lime.*—The lime shall be thoroughly hydrated either by the manufacturer, or the contractor. If hydrated by the contractor, it shall be slaked in sufficient water to make a soft paste and allowed to stand at least one week before being applied to the wall.

4. *Hair or Fiber.*—There shall be used only first quality long cow hair, free from foreign matter, or a long cocoanut fiber well combed out.

5. *Coloring Matter.*—Only inorganic colors shall be used which are not affected by lime, Portland cement, other ingredients of the mortar, or the weather.

6. *Water* shall be clean, free from oil, acid, strong alkalies or vegetable matter.

PREPARATION OF MORTAR.

7. *Mixing.*—The ingredients of the mortar shall be thoroughly mixed to a uniform color, sufficient water added to obtain the desired consistency, and the mixing shall continue until the cement and lime are uniformly distributed and the mass is uniform in color and homogeneous.

The hair or fiber shall be added during the process of wet mixing.

8. *Measuring Proportions.*—Methods of measurement of the proportions of the various ingredients, including the water, shall be used which will secure separate uniform measurements at all times. All proportions stated are by volume. A barrel of cement shall be assumed to contain 3.8 cu. ft. Lime when used shall be measured in the form of putty. Hydrated lime shall be made into putty before being measured.

9. *Quantity.*—There shall not be mixed at one time more mortar than will be used within one hour. Mortar which has begun to stiffen or take on its initial set shall not be used.

10. *Hand Mixing.*—The mixing shall be done on a water-tight platform and the materials shall be turned until they are homogeneous in appearance and color.

11. *Consistency.*—The materials shall be mixed so as to provide sufficient water to insure a proper bonding and a dense mortar free from voids.

12. *Retempering.*—Retempering mortar, *i.e.*, remixing with water after it has partially set, shall not be allowed.

STRUCTURE.

13. *Framing.*—Studs spaced at 12-in. centers wherever possible shall be run from foundation to rafters without any intervening horizontal grain in the wood. These studs shall be tied together just below the floor joists by 6-in. boards which will be let into the studs on their inner side, so as to be flush and securely nailed to them. These boards will also act as sills for the floor joists, which in addition will be securely spiked to the side of the studs.

14. *Bracing.*—The frame of the building shall be so rigidly constructed and braced as to avoid cracking the stucco.

(*a*) At least once between each two floors, brace between the studding with 2 x 3 in. bridging.

(*b*) Bracing may be omitted, as the sheathing boards act as bracing.

15. *Sheathing.*—(*a*) The lath is to be fastened direct to the studding and back-plastered and no sheathing boards are to be used.

(*b*) Sheathing boards shall be not less than 6 in. or more than 8 in. wide, dressed on one or both sides to a uniform thickness of $\frac{7}{8}$ in. They shall be laid diagonally across the wall studs and fastened with two nails at each stud.

16. *Inside Waterproofing.*—(*a*) The faces of the studs and for one inch back of the face on each side where the plaster may come in contact with them, shall be thoroughly waterproofed with tar or asphalt.

(*b*) Over the sheathing boards shall be laid in horizontal layers, beginning at the bottom, a substantial paper well impregnated and thoroughly waterproofed with tar or asphalt. The bottom strip shall lap over the base board at the bottom of the wall, and each strip shall lap the one below at least 2 in. The paper shall lap the flashings at all openings. When required, the lower horizontal edge of each strip shall be cemented with hot or liquid tar or asphalt compound, to the strip below and to the grounds of flashings at all openings. All tacking shall be within 2 in. of the top horizontal edge, where tacks will be covered by the lap of the strip above.

17. *Furring.*—When furring forms a part of the metal lath to be used, then separate furring as described in this paragraph is to be omitted.

(*a*) Galvanized or painted $\frac{1}{2}$-in. crimped furring, not lighter than 22 gage or other shape giving equal results, shall be fastened direct to the studding, using $1\frac{1}{4}$-in. x 14 gage staples, placed 12 in. apart.

(*b*) Galvanized or painted $\frac{1}{2}$-in. crimped furring, not lighter than 22 gage or other shape giving equal results, shall be fastened over the sheathing paper and directly along the line of the studs, using $1\frac{1}{4}$-in. x 14 gage staples, placed 12 in. apart. The same depth of furring should be adhered to around curved sur-

faces, and furring shall be placed not less than $1\frac{1}{2}$ in. or more than 4 in. on each side of and above and below all openings.

18. *Preparation of Original Surface.*—All roof gutters shall be fixed and down-spout hangers and all other fixed supports and fasteners shall be put up before the plastering is done, so there will be no break made in the plastering where they are permanently fixed.

Wall copings, balustrade rails, chimney caps, cornices, etc., shall be built of concrete, stone, tile or metal with ample overhang drip grooves or lip, and water tight joints, to keep water from behind the plaster.

If wood sills are used, they should project well from the face of the plaster and have ample drip groove or lip.

Metal lath shall be stopped far enough above the level of the ground to be free from ground moisture.

Care should be taken to provide for placing all trim the proper distance from the studding or furring to show its right projection after the plaster is on.

19. *Lath.*—The lath shall be not thinner than 24 gage, galvanized or painted, expanded metal lath weighing not less than $3\frac{1}{4}$ lb. to the square yard, or woven wire lath, galvanized or painted, 19 gage, $2\frac{1}{2}$ meshes to the inch, with stiffeners at 8-in. centers.

20. *Application of Lath.*—Place lath horizontally over the furring, driving galvanized staples, $1\frac{1}{4}$-in. x 14 gage, 8 in. apart over the furring into the studding. The sheets of lath shall be locked or lapped at least 1 in. and tied at joints between studs both vertically and horizontally with 18 gage wire.

21. *Corners.*—There shall be 6-in. strips of metal lath bent around the corners and stapled over the lath, or the sheets of metal lath shall be folded around the corners a distance of at least 3 in. and stapled down, as applied. Galvanized corner bead may be applied over the lath.

22. *Insulation.*—(*a*) After the lath on the outside has been back-plastered, the air space may be divided by applying heavy building paper, quilting, felt or other suitable insulating material between the studs, fastening it to the studs by nailing wood strips over folded ends of the material. This insulation should be so fastened as to clear the bridging, leaving the preponderance of the air space next to the plaster. Care must be taken to keep

the insulating material clear of the outside plaster and to make tight joints against the wood framing at the top and bottom of the spaces and against the bridging where the face intercepts.

(*b*) When quilting, felt, or other insulating material is to be used it shall be applied to the sheathing boards under the inside waterproofing.

23. *Brick, Tile or Cement Block Surfaces.—Existing surfaces* to be stuccoed shall have all loose, friable or soft mortar removed from the joints to a depth of not less than $\frac{1}{2}$ in. All dirt, dust or any other foreign matter shall be removed by means of a wire brush, stiff broom or other effective means. In case the surface has been painted, is oily or otherwise in such condition that the stucco will not firmly adhere, then metal furring and lathing shall first be applied.

New surfaces shall have ample roughness to assure a strong bond and key between the stucco and the surface. The mortar joints shall not be less than $\frac{3}{8}$ in. thick and the mortar shall be omitted from or raked out of the joints for at least $\frac{1}{2}$ in. back from the face to which the stucco is applied. Before placing the scratch coat the surface shall be brushed clean from all dust, dirt or loose particles and thoroughly wetted.

Mortar Coats.

24. *Plaster.*—(*a*) The *first coat* shall contain not more than two and one-half ($2\frac{1}{2}$) parts of sand to one (1) part of Portland cement by volume. If lime putty is added, it shall not be in excess of one-third ($\frac{1}{3}$) of the volume of cement. Hair or fiber may be added in sufficient quantity to bond the mortar.

(*b*) The *first coat* shall contain not more than two and one-half ($2\frac{1}{2}$) parts of sand to one (1) part of Portland cement by volume. If lime putty is added it shall not be in excess of one-third ($\frac{1}{3}$) of the volume of cement. No hair, fiber or similar material of any kind or in any quantity shall be added to the mortar.

For *second coat,* the proportion of sand to cement shall not be greater than $2\frac{1}{2}$ to 1 by volume, nor shall more than $\frac{1}{3}$ part of lime putty be added.

For *third coat,* the proportion of sand to cement shall not be less than 2 to 1 nor more than $2\frac{1}{2}$ to 1, by volume, nor shall more than $\frac{1}{3}$ part of lime putty be added.

25. *Application.*—The plastering should be carried on continuously in one general direction, without allowing the plaster to dry at the edge. If it is impossible to work the full width of the wall at one time, the joint should be at some natural division of the surface, such as a window or door.

Metal Lath.—(*a*) The first coat shall be applied to the lath and pushed through sufficiently to give a good key. Over the face of the studs the plaster shall be forced well through the lath in order to fill entirely the space between the lath and the stud. The backing coat shall be applied to the back of the lath and shall be thoroughly troweled so that the lath shall be entirely covered. The final coat shall be applied to the face of the first coat.

(*b*) The first coat shall be applied to the lath and thoroughly pushed through against the inside waterproofing so as to completely imbed the metal of the lath on both sides. Special care shall be taken to fill all voids around furring strips and where laths lap.

Brick, Tile or Cement Block Surfaces.— The first coat shall be forcibly and thoroughly troweled into the depressions of the previously saturated surface so as to make a firm bond. Care shall be taken to insure the complete filling by the mortar of all crevices and pores.

If subsequent coats are applied they shall be well troweled to insure good bond with the previous coat.

26. *Roughing.*—Soon after applying and before the initial set has taken place, the surface of the coats which are to receive succeeding coats shall be roughened with a saw-toothed paddle or other suitable device.

27. *Dampening.*—Before applying mortar the surface of the preceding coat shall be thoroughly wetted to prevent absorption of water from the fresh mortar.

28. *Thickness of Coats.*—(*a*) The first coat shall be at least $\frac{3}{8}$ in. thick over the face of the lath and project through behind the lath about $\frac{3}{8}$ in. The backing coat shall increase the thickness behind the lath to not less than $\frac{5}{8}$ in. The final coat shall be not less than $\frac{3}{8}$ in. thick.

(*b*) The first coat shall have a minimum thickness over the lath at any point of not less than $\frac{1}{4}$ in. The intermediate coat shall have a thickness of not less than $\frac{1}{4}$ in. or more than $\frac{3}{8}$ in.

The final coat shall have a thickness of $\frac{1}{4}$ in. when placed over an intermediate coat, or of $\frac{3}{8}$ in. when placed directly on the scratch coat.

29. *Drying Out.*—The final coat shall not be permitted to dry out rapidly, and adequate precaution shall be taken, either by sprinkling frequently after the mortar has set hard enough to permit it or by hanging wet burlap or other material over the surface.

30. *Freezing.*—Stucco should never be applied when the temperature is below freezing.

FINISH.

31. *Smooth Troweled.*—The finishing coat shall be troweled smooth with a metal trowel with as little rubbing as possible.

32. *Stippled.*—The finishing coat shall be troweled smooth with a metal trowel with as little rubbing as possible, and then shall be lightly patted with a brush of broom straw to give an even, stippled surface.

33. *Sand Floated.*—The finishing coat, after being brought to a smooth, even surface, shall be rubbed with a circular motion of a wood float with the addition of a little sand to slightly roughen the surface. This floating shall be done when the mortar has partially set.

34. *Sand Sprayed.*—After the finishing coat has been brought to an even surface, it shall be sprayed by means of a wide, long fiber brush—a whisk-broom does very well—dipped into a creamy mixture of equal parts of cement and sand, mixed fresh every 30 minutes and kept well stirred in the bucket by means of the whisk-broom or a paddle. This coating shall be thrown forcibly against the surface to be finished. This treatment shall be applied while the finishing coat is still moist and before it has attained its final set, *i.e.*, within 3 to 5 hours. To obtain lighter shades, add hydrated lime of 5 to 15 per cent of the volume of the cement.

35. *Splatter Dash or Rough Cast.*—After the finishing coat has been brought to a smooth, even surface and before attaining final set, it shall be uniformly coated with a mixture of one part cement and two parts of sand thrown forcibly against it to pro-

duce a rough surface of uniform texture when viewed from a distance of 20 ft. Special care shall be taken to prevent the rapid drying out of this finish.

36. *Pebble Dash.*—After the finishing coat has been brought to a smooth, even surface, and before attaining initial set, clean round pebbles or other material as selected, not smaller than $\frac{1}{4}$ in. or larger than $\frac{3}{4}$ in., previously wetted, shall be thrown forcibly against the mortar so as to imbed themselves in the fresh mortar. They shall be distributed uniformly over the surface of the final coat and may be pushed back into the mortar with a clean wood trowel, but no rubbing of the surface shall be done after the pebbles are imbedded.

37. *Exposed Aggregates.*—The finishing coat shall be composed of an approved, selected coarse sand, marble dust, granite dust or other special material, in the proportion given for finishing coats, and within 24 hours after being applied and troweled to an even surface, shall be scrubbed with a stiff brush and water. In case the cement is too hard a solution of one part hydrochloric acid in four parts of water by volume can be used in place of water. After the aggregate particles have been uniformly exposed by scrubbing, particular care shall be taken to remove all traces of the acid by thorough spraying with a hose.

38. *Mortar Colors.*—When it is required that any of the above finishes shall be made with colored mortar, not more than 6 per cent of the weight of Portland cement shall be added to the mortar in the form of finely ground coloring matter.

A predetermined weight of color shall be added dry to each batch of dry fine aggregate before the cement is added. The color and fine aggregate shall be mixed together and then the cement mixed in. The whole shall then be thoroughly mixed dry by shoveling from one pile to another through a $\frac{1}{4}$-in. mesh wire screen until the entire batch is of uniform color. Water shall then be added to bring the mortar to a proper plastering consistency.

MACHINE STUCCO.

39. Stucco may be applied by a machine provided the results obtained are equal to those produced by hand work.

OVERCOATING.

During recent years there has come into vogue a method of remodeling old frame houses. This "overcoating," as it is called, is used extensively in all sections of the country and the following practice is recommended:

40. Where furring is used so deep that the space back of the lath is not entirely filled with plaster, some provision must be made for extending the old window and door frames to correspond with the increased thickness of the wall. In some cases the plaster is brought over the old frames in such a manner that a recessed window or door opening is made. In case the furring is fastened to the studding, it is not necessary to provide for extending the window and door frames, as the new stucco finish will have the same relations as the old weather-boarding.

41. *Preparation of Original Surface.*—If the weather-boarding is in poor condition it should be removed and furring and metal lath applied over the sheathing, to which waterproof paper has previously been fastened. It may be advisable also to tear off the sheathing, in which case the furring can be fastened direct to the studding after bracing between the studs.

Another method would be to fasten the furring direct over the weather-boarding over which the metal lath is applied.

In preparation for any of these methods the house should be gone over carefully to determine if the framework is well enough preserved to justify the improvement.

The doors should be looked after, the studding inspected, partitions and outside walls lined up and brought into plumb.

42. *Furring.*—Fasten galvanized or painted $\frac{1}{2}$-in. crimped furring or other shape giving equal results, vertically over the original surface, which ever of the above may be adopted.

43. *Lathing and Plastering.*—Follow the above specifications for stucco.

DISCUSSION.

Mr. McMaster. MR. H. B. MCMASTER.—So far as our association is concerned this specification meets the approval of the manufacturers of metal lath in the United States. In exploiting the specifications we will probably eliminate those features dealing with competing construction. We recognize, of course, that this Association as well as the cement manufacturers must take a broader view, but in providing for the use of cement in stucco work, we are providing particularly for the use of metal lath.

I take this occasion to express my appreciation of the opportunity for co-operating in the very valuable work which this committee has done so very well.

Mr. Ferguson. MR. L. R. FERGUSON.—In regard to the protection of wooden lath, I would suggest instead of painting both sides, as proposed by Mr. Wason, that the lath be dipped in bitumen paint, which would be cheaper and more satisfactory.

This was done in Baltimore, but the time has not been sufficient to determine whether or not it will be successful. The application of stucco to wooden lath is not successful; this may be due to the fact that it absorbs moisture either from the front or back; probably the latter, because of the greater difficulty in securing a proper embedment around the wide face of the wooden lath, as compared to metal lath.

I believe all other points under discussion between the Association of American Portland Cement Manufacturers and this Association have been covered.

Mr. Boyer. MR. E. D. BOYER.—Mr. President, I do not quite agree with this specification for stucco on wood lath. I do not believe we know enough about stucco on wooden lath to publish a specification for it. Personally I have never seen stucco on wood lath that has been successful. I have seen a whole lot of it that has been unsuccessful. I understand that down in Mississippi, where they have a lath made of special wood, it has been used with some success; but I do not know of any cases in the northern states where it has been used successfully.

I surely think that wood lath, if it is protected by a coating, will not make the right sort of bond, and I believe unquestionably there will be cracking where a thin surface is plastered on to the lath. Then there is danger of staining through the thin coat of stucco, when a wooden lath is coated with bitumen or tar, especially where a white effect is desired. Mr. Boyer.

PRESIDENT HUMPHREY.—It certainly does not seem wise for this Association to adopt a specification for stucco on wood lath when there are many examples of its unsuccessful application, but none to show that it has been successfully applied. Pres. Humphrey.

It would be wiser to publish the specification with this discussion as information.

MR. CLOYD M. CHAPMAN.—I think the chief advocate of stucco on wooden lath among the members of this Association has been Mr. Spackman; he states that he has used it successfully, prefers it to any other and doesn't use anything else in his own practice; he is the author of last year's wording of the specification for stucco on wood lath. I think he was the only member of the Association, as far as I recall, who strongly advocated the use of wood lath when the matter was discussed at previous meetings. Mr. Chapman.

Creosote is a fairly good waterproofer, and a good preserver, but it is penetrating and it will work into the stucco as well as into the wood. I am convinced it is not a suitable treatment for wood which is to be covered so lightly with stucco, that it is very liable to stain. I move that the word "creosote" be stricken out'

PRESIDENT HUMPHREY.—The ayes have it, the motion is carried. Pres. Humphrey.

MR. JOSEPH H. CHRISTOPHER.—I have used stucco in all ways in the last fifteen years in building stucco houses. The first house in my town was stucco on expanded metal and it is in good condition today. There is also one built of stucco on wood lath that has gone to pieces. In the last five years we have placed considerable stucco on cement blocks, and there is also considerable of it being placed on hollow tile. This is considered the best way to put on stucco, and it is felt that it makes a permanent building. The shrinkage of timber causes cracks in the stucco and this permits water to enter which rusts the iron or rots the wood; the stucco falls off and leaves the building appearing worse than one of frame. Mr. Christopher.

Mr. Christopher. I will speak of a stucco house built for myself, where wire mesh or wire cloth was used, and the specifications adopted a few years ago were followed as closely as possible. The work was done by day's labor, there was no need for skimping, and I think it was done well. I observed only the other day a crack in the cement running across it; I observed that defect in more than one place, and the general impression that I have is that sooner or later there will be a lot of cheap looking houses throughout the country if we are not careful; this Association should be cautious in making recommendations for stucco on any material.

Pres. Humphrey. PRESIDENT HUMPHREY.—In accordance with the vote just taken the Committee will eliminate stucco on wood lath from these specifications.

Mr. Wig. MR. R. J. WIG.—This Association has never made a recommendation in the matter of waterproofing; I move that this paragraph (24) be stricken out.

Pres. Humphrey. PRESIDENT HUMPHREY.—The ayes have it, the motion is agreed to.

Mr. Schouler. MR. W. W. SCHOULER.—Mr. President, it has been my observation that frequently cracks and the apparent breaking up of the metal or wood lath are caused by improperly hydrated lime; and I do not allow my men to use any lime at all in what little stucco work I do. I have seen the whole outside of a house peel off because the lime was not thoroughly hydrated. Some hydrated lime men do not recommend their material for the outside of a house where it is exposed to temperature changes. I have seen stucco go to pieces, not only where lime was used, but also where water entering near the top of a building penetrated the inside and rusted every bit of the iron, leaving the cement that finally cracked and was thrown off, I presume, by frost action. On part of a frame house, the stucco which I applied on wire mesh, furred out flat, under the architect's direction, came off. There was no protection at the cornice and water got in, in some unaccountable way.

Mr. McMaster. MR. MCMASTER.—We have with us Mr. Bullard, of Boston, who is one of the pioneers in metal lath manufacturing, and I wish he would give us the result of his experience in Boston, particularly with stucco work of the kind Mr. Christopher spoke

about; that is, metal lath with plaster applied to both sides, and particularly without sheathing. Mr. McMaster.

Mr. George P. Bullard.—I have had a rather long experience in stucco construction and have been in position to notice carefully a great deal of completed work. You all know that we have a trying climate in New England. The sea exposure of our north shore is about as bad as possible, and there deterioration would be about as liable as in any place. Mr. Bullard.

I have in mind the residence of Mr. H. P. McKean at Prides Crossing, on the north shore above Boston, a particularly bad exposure on a hill, that was not built in accordance with the specifications that have just been discussed, but was an ordinary construction of weather board, to which was applied expanded metal of the old type. It became necessary to make certain changes and additions, so that a portion of the building was torn down. The contractor was pleased, as well as surprised, to find that after fourteen years this lath was in as good condition as when it was put on. He gave me a number of samples which I have in my office.

I built a residence, some six years ago in West Newton, Mass., in accordance with the specification which we favor; that is, no weather boarding, the lath applied to the studs, set 12 in. apart, using ordinary unpainted lath; first there is applied on the outside, a heavy scratch coat of lime mortar with plenty of hair, then a heavy backing coat is applied on the inside between the studs, and the browning and finishing coats on the outside with Portland cement and sand, constructing in this way a 2-in. solid cement wall. The interior of the house was all three-coat work on expanded metal. The roof was flat tile, so that in the construction there was not used a weather board, clapboard, shingle or wood lath. Experience proves that a house so constructed is cool in summer, warm in winter, and is in every way admirable.

I have also in mind a building which was built about ten years ago on the Nantasket Beach Reservation under a similar specification for the stucco work. I see that building every summer and always take particular pains to examine it before I leave; last year I could discover absolutely no cracks, nor the slightest evidence of disintegration. In fact, the building, so far as the plaster was concerned, appeared to be in exactly the same condition as when the plasterer left it.

Mr. Bullard. Now these and many other cases with which I am personally familiar indicate that in spite of what has been said there are very many satisfactory examples of stucco construction. I believe the specifications that have been advocated will give satisfactory work if they are well and honestly carried out by competent plasterers.

Mr. Ferguson. MR. FERGUSON.—In reading these specifications more carefully I find they do not agree. They provide for 1 part of cement to 5 parts of sand and about 30 per cent of lime, which is entirely too much sand to make a dense mortar. The total amount of sand should not exceed $2\frac{1}{2}$ parts; 30 per cent lime is high, it should be not more than 20 per cent.

Mr. Chapman. MR. CHAPMAN.—In my opinion the amount used should be $2\frac{1}{2}$ parts to 1 of cement, and the lime in the mixture should never be over 25 per cent by weight of the cement. I have talked with a number of people who advocate the use of a straight lime mortar, for the first coat, placing over this a cement finishing coat. This practice seemed to have so many advocates that in drawing up this specification we put in considerably more lime that we would specify in our own practice. We would use about $12\frac{1}{2}$ to 15 per cent.

Mr. Arp. MR. C. K. ARP.—I do not agree with Mr. Chapman that lime may be used in the scratch coat. The lasting qualities and strength of Portland cement stucco, on either metal or wood lath, depend largely upon the strength of the key which the foundation or scratch coat makes with the framework on which it is applied. Lime, when used in stucco work, is only added to render the mortar more plastic, thereby enabling the workman to apply it rapidly and easily. Lime, however, when added to cement-sand-mortar, has a tendency to reduce the strength of the key. It is, therefore, advisable to eliminate it, at least in the scratch coat. A well-known plaster contractor in Chicago does not permit the use of lime in the scratch coat, or even in the succeeding coats, but specifies that lime water be used in gaging the cement-sand-mortar and claims that a mortar made in this way is sufficiently plastic to enable his men to apply the mortar easily.

Mr. Boyer. MR. BOYER.—Mr. President, I do not agree with these mixtures for stucco. I agree with the statement that the first coat must contain Portland cement. The stucco specification that

the company I represent is using requires Portland cement with the addition of 20 per cent of hydrated lime. Lime putty is hard to mix; hydrated lime answers the same purpose and it is much easier to handle. Mr. Chapman says hydrated lime will pit; hydrated lime properly hydrated, will not pit. Mr. Boyer.

Mr. Schouler.—I suggest that the committee insert a section in the specification eliminating lime altogether. I am a cement contractor and never allow a particle of lime to go in any face work or splatter-dash work. We are afraid of it, for we never know when it is hydrated. You should not use lime putty three days old. The manufacturers do not recommend hydrated lime on the outside of a building, or any place where water can get at it; the swelling of the lime will disintegrate and crack the cement. I have seen many examples of concrete work spoiled through the use of lime. If we could get masons to use splatter-dash, we could teach them to eliminate lime. It seems to me that the specifications should provide for splatter-dash work, and should eliminate lime. Mr. Schouler.

I have seen lime putty thirty days old that would pop in the wall. There is a gentleman here from Wisconsin, an experienced contractor and cement worker, who will tell you that he never uses a particle of lime in splatter-dash.

Mr. George V. Thompson.—If I was to put a plaster on the outside of a building I would not use lime. Mr. Thompson.

Mr. Chapman.—The committee has considered the elimination of lime, but it feels that the possibility of inducing stucco workers in general to use a straight cement-sand mixture is rather slim. Plasterers object very strenuously to it—they would prefer to mix half lime and half cement. The committee desires to recommend a specification that will be acceptable. As to objection to mortar being made up only three days; the specification states in another paragraph that the lime shall be slacked a week. This three days applies to the period that the sand shall be mixed with the lime putty which has been slacked a week previous. I think it would be well if Mr. Schouler would make a motion covering his suggestion as to the alternate specification for cement mortar coats in place of cement lime mortar coats. Mr. Chapman.

Mr. Schouler. MR. SCHOULER.—I move that there be incorporated in the specifications a provision for using cement alone instead of cement and lime. The proportions should be 2 : 1 for scratch-coats with hair in proper amount, 3 : 1 for float finish, and 2 : 1 for a splatter-dash. I do not think it is necessary to put hair in the float finish above the scratch-coat. Three to one is too lean for the second coat because it is manipulated with the float, whereas the outside is thrown on; anything that you throw on eliminates the air and stays where it is placed. When I plaster a reinforced wall, I throw the scratch coat on as though I was making a splatter-dash, and then it stays. When you put it on with the trowel you get air between it and the wall and you have no assurance that it will stay. We use a 3 : 1 second coat, and it works all right; it is a strong mixture, but it does not protect the lath from the moisture and the 2 : 1 mixture on the outside keeps the rain from penetrating to the lath.

Mr. Chapman. MR. CHAPMAN.—The splatter-dash finish is only one of many finishes that the committee specifies, and in some localities it is very little used because of its ability to gather dirt and become dingy. So that as far as this specification is concerned it is not a part of the principal coating, but is one that may be applied over the principal coat. The main coats are the scratch, the second and the finish, which in the case of one particular finish, is the splatter-dash coat. The committee has been for the last three years in communication with practical stucco men in New York and New England who are doing this work, getting suggestions and discussing the question with them, both non-members and members. The committee has tried to gather the best from all sources.

Mr. Boynton. MR. C. W. BOYNTON.—Will the chairman of the committee please state the attitude of the committee with reference to wood lath? Personally I favor metal lath though I think some work put on metal lath is no better than that put on wood lath. Some architects have become dissatisfied with the results obtained with metal lath; therefore I feel this Association should not take a definite stand at this time against the use of wood lath.

Mr. Chapman. MR. CHAPMAN.—The specification has always included a section on wood lath. The former specification stated that wood lath should be wet before being plastered. The Association of

American Portland Cement Manufacturers offered an amendment that wood lath be painted with two coats of a non-absorbent paint, such as tar or asphalt, before the mortar was applied. This Convention is not at present in favor of wood lath. While wood lath is used and there are members who would uphold it, they are not here to defend it. The members present, including the committee, do not feel qualified to criticise the wood lath specification. They are inclined to doubt the value of wood lath, and in order to get at least a specification for metal lath, it was decided to eliminate the wood lath section. This action now leaves the way open for another specification covering wood lath which could later be combined with this one. Mr. Chapman.

MR. SANFORD E. THOMPSON.—This is an extremely important point. I have known of a number of cases where metal lath has rusted so as to actually fall down. In other cases, where the wall has been cut into, it was found in such condition as to rust that it could not have stood much longer. I do not feel that we know any more about the durability of plaster on metal lath than we do of work on wood lath. When the time comes, I propose to present a motion that the committee be instructed to investigate the wearing qualities of both materials. Mr. Thompson.

TESTS OF WATERPROOFING FOR CONCRETE.

By Cloyd M. Chapman.*

So much that is contradictory has been said and written concerning various methods of producing concrete which is waterproof and so much has been claimed by the vendors of various compounds which have been offered to the public as a positive solution of all waterproofing problems, that it is not surprising that not one cement user in ten has any definite or fixed opinion as to how to meet the problem of producing a waterproof concrete. There seem to be as many opinions on the subject as there are men willing to express their opinions. The many methods which have been used differ greatly from each other; they are not all bad, for some of them have been very successfully applied; they are not all good, for many failures have occurred. Every little while a new preparation is offered or a new method is advocated.

There seems to be no strong tendency toward any one method of producing a waterproof concrete. Perhaps the reason for this continued uncertainty and diversity of opinion among cement users may be found in the very fact that many of the multitude of methods advocated have, in certain instances, proven successful and each success makes converts to the method used. In many cases where special compounds have been added to a good, well-proportioned concrete made of properly-graded materials, there is little doubt that the result would have been satisfactory if the special compound had been entirely omitted. In other cases where the concrete is made of materials not wholly adapted to the purpose the addition of a suitable filler or special compound would have produced the desired waterproof qualities.

To throw more light upon the effectiveness of the various methods of waterproofing concrete a great many tests have been made by numerous investigators. The methods used by these experimenters have ranged from absorption tests to high-pressure tests, necessitating more or less elaborate apparatus.

The absorption or non-pressure tests are made by preparing

* Engineer-in-charge, Westinghouse, Church, Kerr and Company, New York, N. Y

test pieces of any predetermined size and shape of the concrete, waterproofed by the particular methods under investigation. After the desired age is attained these test pieces are either immersed in water completely, or immersed almost completely or placed in shallow water so that the bottom only of the test piece is wet, or the same may be made cup-shaped so that the water may be placed in the depression in the test piece, or a container for the water may be attached to its surface. The two last mentioned forms of tests have been extensively used in the laboratory of Westinghouse, Church, Kerr and Company. In all of these forms of tests the results are expressed in terms of the amount of water absorbed, or the rate of absorption.

In the pressure tests the test piece is either so made that the water pressure may be applied to the interior of the test piece or so that it may be inserted in the apparatus in such a manner that the water pressure may be applied to a certain area on one face of the test piece. In this form of test the results are expressed in terms of the amount of water passing through the test piece or the rate of flow through it.

Both of the above classes of tests are very valuable if carried out in such a manner that the results may be reasonably expected to compare with results which may be obtained in practical work. To this end it is desirable that certain precautions be observed in preparing test pieces and conducting tests so that the results shall not be misleading or deceiving.

It is the purpose of this paper to call attention to some of the features of tests of this class which should receive particular attention and to emphasize more strongly certain precautions, failure to observe which may cause some of these tests to give very deceptive data. These precautions apply particularly to tests made for the purpose of determining what method shall be used on a particular piece of work in the field. Some of them do not apply to tests designed only to compare the relative efficiency of various methods without regard to any particular work at hand.

PREPARATION OF TEST PIECES.

1. *Selection of Materials Used in Making Test Pieces.*—These should be selected from those available at the site of the work which is to be done. This is particularly true of the fine and coarse

aggregates. It has more than once happened that tests made with suitable aggregates have shown a particular method to give a waterproof concrete but the same treatment failed when the aggregates available in the field were used. If, therefore, it is desired to determine by test how to produce a waterproof concrete for a particular job, the materials available at the job should be used in making up the test pieces. It is also important in connection with the selection of aggregates that there be secured a sufficient quantity of the materials to be used to conduct the entire series of tests contemplated, so that there need be no change in the materials used.

2. *Proportioning the Materials.*—This should be done with a view to determining the proportions to be used later on the work. The proportions to be used on the job should be used in the tests. There is little use in testing only a 1:2:4 concrete for water-tightness and then use $1:2\frac{1}{2}:5$ on the job.

3. *Mixing the Materials.*—This should duplicate as nearly as possible the mixing to be used on the work. Many a well-mixed concrete has proven waterproof which would have failed utterly if carelessly mixed. There is liable to be a tendency to very thoroughly mix concrete for a test and then to make no special provision for thorough mixing on the job.

4. *Consistency of the Mixture.*—This should be carefully considered because of its influence upon the results. A consistency should be chosen which it is practical to use on the job under the circumstances prevailing there. For instance, it is useless to test a concrete of so thick a consistency that it would have to be spaded into the forms if, on the job, the concrete is to be spouted from the elevator and therefore necessarily must be of a much thinner consistency than used in the test.

5. *Molding the Test Piece.*—This process should imitate as nearly as possible the filling of the forms as it will be done on the job. To make a test piece by putting into the mold a small quantity of concrete at a time and constantly tamping during the filling process would probably produce a deceiving result.

6. *Finishing the Surface to be Tested.*—This is one of the most important points in the preparation of test pieces. The condition in which the surface to be subjected to water is left has a very great influence upon the results obtained. For instance, it is

quite possible by means of a little troweling to obtain a skin or surface coating of nearly neat cement. This will produce a dense surface much more impermeable than the body of the concrete and show results far superior to an untroweled test piece of the same concrete. It is recommended that test pieces be so prepared that the surface exposed to test truly represents the body of the concrete. This may be accomplished by removing the surface of the test piece after the concrete has hardened by means of a wire brush or by chipping or breaking off the surface before test.

In the case of absorption tests, this may be done with a stiff wire brush when the test piece is about twenty-four hours old. In the case of the pressure tests only that portion of the surface which is to be subjected to the pressure and that portion from which the water passes out of the test pieces needs to be removed. This may be accomplished by making the test piece with additional material added to the surfaces to be tested so that this additional concrete may be broken off after hardening and before the test is performed. Unless some such precaution is taken tests of this character are liable to show the efficiency of the surface of the concrete in resisting the water pressure rather than the value of the mass of the concrete in performing the same function.

One suitable form for a test piece for pressure tests is shown in Fig. 1. It is in the form of a central disc *a* of suitable diameter and thickness. The outside surface and the outer portion of the two faces of this disc are molded smooth and regular so as to fit the receptacle or holder into which it must be placed in order to apply the water pressure to a definite and restricted area of its surface. From the central portion of the two faces of this disc project truncated cones *bb* whose smaller diameter is just equal to the diameter of the circle which is to receive the water pressure. After the test piece has hardened and is ready for test, the two truncated cones are broken off with a blow of a hammer leaving a disc such as is shown at *c*.

By this method there is exposed to the water pressure a freshly broken surface of concrete which has not been subjected to troweling or other influences tending to alter the natural condition or distribution of the constituents of the concrete. This freshly broken surface is not only presented to the water

on the side to which the pressure is applied, but is also provided on the opposite side where the water leaves the test piece. The test, therefore, is made on a certain thickness of concrete taken from the interior of the test piece and the results are not influenced by the method of finishing these two surfaces.

There are, of course, cases in which it is desired to test the surface permeability and in such cases this form of test piece is unsuitable. But in all cases in which the water-tightness of the concrete mass is to be determined the above type of test piece is much to be preferred.

There are, also, variations of this form of test piece which will accomplish the same result. Any method which removes the finished surface from the area to be tested is to be preferred to one which does not remove that surface. Even in such cases

FIG. 1.—TEST PIECE FOR PRESSURE TEST.

as those in which the test piece is molded on a surface of glass or other smooth material and no troweling is done, there is a concentration of cement and fine aggregates next to the surface which must affect the results obtained if this richer layer or skin is not removed before test.

7. *Curing or Ageing of Test Piece.*—This matter should also be decided with a view to conditions which will prevail in the field when the real work is done. If it is unpracticable on the job to keep the concrete constantly wet for a considerable period, then the test pieces should not be stored in water. If the only wetting the work in the field is to receive is the water contained in the concrete when it is placed, then the test pieces should receive no additional water after molding. On the other hand, they should not be allowed to dry out any faster than would the work in the field, as they would doubtless do owing to their comparatively

small size if kept indoors in a laboratory and no precautions taken to regulate the drying process.

Tests.

The test pieces have now reached the stage when they are ready for whatever tests are to be applied to them and the methods used in performing these tests are so varied that the limited scope of this paper will not permit an extended discussion of them. Those with which the writer is familiar appear to be open to little, if any, criticism. One of the most important features is that the conditions adopted shall remain constant throughout the test.

If the test is an absorption or a non-pressure test, the immersion or partial immersion should be under uniform conditions and for definite lengths of time in clean water. If the test is a pressure test, the pressure should be kept constant, the water clean and the method of measuring the water passing through the concrete accurate. As the area subjected to pressure is usually small, the amount of water passing is correspondingly small and in some cases where the measurement of the water is made by collecting the drippings from the under side of the test piece the element of evaporation may greatly affect the results. A form of apparatus in which the amount of water passing into the specimen is determined eliminates this error.

In this form of apparatus, Fig. 2, the test piece is clamped and sealed to a metal cap which is provided with a projecting, vertical, graduated glass tube. The cap and the tube are filled with water and air pressure is then applied to the top of the tube in any desired amount. As soon as the test piece has become saturated and water begins to flow from its exposed face, a reading is taken of the water in the graduated glass tube and thereafter readings are taken at regular intervals of time during the period of the test. If it is necessary to introduce more water into the system when testing porous concretes, this may be done by opening an inlet which is supplied with water at a higher pressure than that of the air in the tube and so filling the glass tube again up to the zero mark.

By this method accurate determination may be made of the amount of water forced into a concrete which is so near waterproof that all, or a great part, of the water which passes through it would be evaporated from the exposed surface.

Another matter in connection with tests of waterproofing for concrete which seems to have had but little attention paid to it, is the effect of time and the elements upon efficiency of the

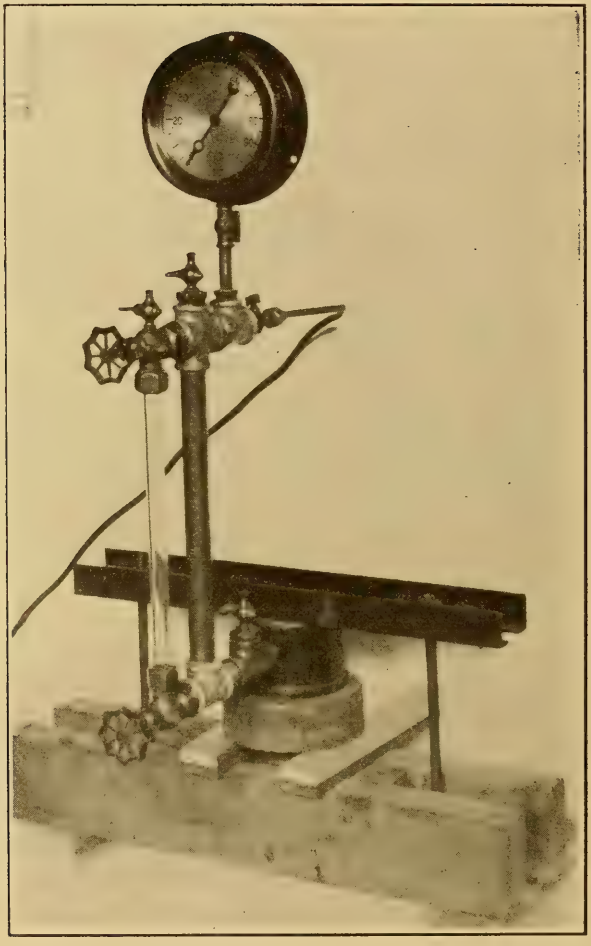


FIG. 2.—APPARATUS FOR DETERMINING PERCOLATION.

waterproofing materials. In practically all of the numerous tests of waterproofings made in the past seven or eight years in the laboratory of Westinghouse, Church, Kerr and Company, it has

been the custom to expose the test pieces to the action of the weather on the roof of their office building after first testing them and then testing again after six or twelve months' exposure. The results of these tests after prolonged exposure show that few, if any, of the materials which are applied to the surface of concrete to waterproof it after it is made, will retain even a fair proportion of their efficiency. In the case of those methods by means of which the entire mass of the concrete is designed to be waterproof, there is shown sometimes a steady improvement after exposure and sometimes a marked decline. In some cases the life of the treatment is very short and the failure after a few months' exposure almost complete.

It is important, therefore, before any particular method of waterproofing be adopted, that the probable life of the treatment be ascertained. It is pretty well established that a good concrete without foreign substances in it improves with age, becomes more dense and water-tight, but the same cannot be said as positively of a concrete containing some of the recently developed compounds intended for waterproofing.

THE USE OF CEMENT ON NATIONAL IRRIGATION WORKS.

By Arthur P. Davis.*

The art of irrigation is older than history, even in the United States. The remains of ancient canal systems used by the aborigines are plainly evident in the valleys of Southern Arizona and elsewhere. Modern irrigation on a considerable scale was first practiced by the Mormons in Utah and by various small colonies in other localities. The early settlers turned water from the small streams upon the adjacent valley lands, where it could be done often with no diversion dam at all, or with the crudest sort of shoal formed of rocks and brush. In many cases no headgate was provided and the development was limited to lands of good slope lying immediately contiguous to the streams. Such work, of course, costing nothing except the farmer's time, could only be effective in limited localities. Where larger streams and larger canals were required, the association of a few farmers, with limited expenditure for materials and equipment, accomplished large results where the circumstances were favorable, and in this way for years irrigation gradually grew to extensive dimensions with very slight expenditure per acre.

As these early favorable locations were utilized, more and more difficult enterprises were developed, requiring heavier construction work and more investment of capital. In most cases this was carried on by co-operative effort and the results were usually gratifying, though often reached through much effort and tribulation.

As the problems presented became progressively more expensive, private capital was interested on a large scale and efforts were made to pay interest on such investments from the results of irrigation. The investment of capital as such in this work has, with rare exceptions, proved unprofitable owing to the slow development of large areas under irrigation and the accumu-

* Chief Engineer, U. S. Reclamation Service, Washington, D. C.

FIG. 1.—SHOSHONE PROJECT, WYOMING. SHOSHONE DAM, DOWNSTREAM FACE.

lation of interest and operation costs during the process of settlement and subjugation of the soil.

As the engineering problems became more and more difficult with progressive development, various laws were passed designed to encourage the investment of private capital in irrigation works. On August 18, 1894, the President approved the so-called Carey Act, which provides machinery for the reclamation of public lands under the authority of the state with certain supervisory powers in the Secretary of the Interior.

The purpose of this act was to devote the value of the public lands after irrigation to the repayment of the capital invested in their reclamation and thus encourage private and corporate

FIG. 2.—NORTH PLATTE PROJECT, NEBRASKA-WYOMING SPRING CANYON FLUME, INTERSTATE CANAL.

investment in this direction. Although this act has been in force about eighteen years, only a few of the states have accomplished any reclamation under its provisions, the total area reclaimed thereunder being less than 300,000 acres.

In view of the engineering difficulties of further large irrigation development, Congress passed an act in 1902 devoting the proceeds from the sale of public lands to irrigation construction. The Reclamation Service was organized in the Department of the Interior for the prosecution of this work and has expended to date about $70,000,000 in the construction of irrigation works.

Since its organization the Reclamation Service has used about 1,500,000 barrels of Portland cement, representing about as many cubic yards of concrete. It has built about 7,000 miles of

canals, 69 tunnels aggregating 111,000 ft.; has built 18 storage dams (Fig. 1) and 25 diversion dams; about 2,000 concrete structures upon canals, and about 32,000 wooden structures. It has built altogether 2,908 bridges with an aggregate length of about 62,000 ft. It has built 436 offices and dwellings and about as many other buildings used for barns and store-houses. It has constructed over 2,000 miles of telephone lines and has over 800 telephones in use. The total excavation of earth and rock is nearly 100,000,000 cu. yd. These works have provided a water

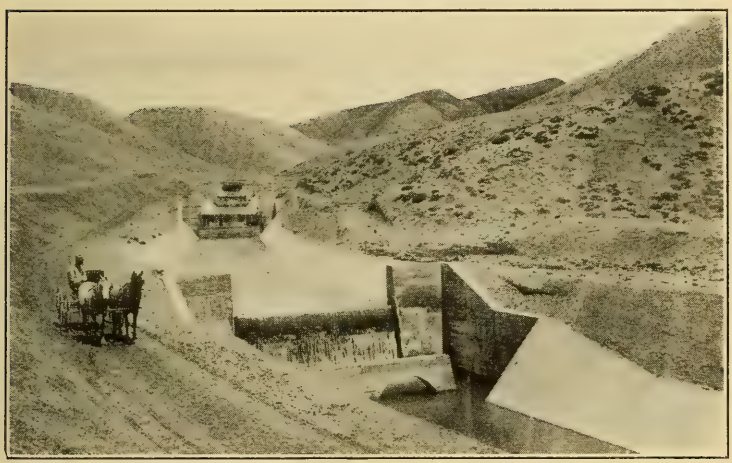


FIG. 3.—UNCOMPAHGRE PROJECT, COLORADO. SERIES OF DROPS IN THE SOUTH CANAL.

supply for about 1,160,000 acres, about one-half of which was irrigated during the past season by about 15,000 families. The projects now under construction, when completed, will supply water to over 3,000,000 acres of land.

Practically all of these works require large storage reservoirs, long and expensive tunnels, flumes (Fig. 2), pressure pipes and other expensive structures for carrying water through mountainous country to high bench lands, and, most important of all, many miles of large canals must be carried upon steep side hills, which present peculiar difficulties both in construction and in maintenance. On such works as these it is hardly necessary to

state that the most important material of construction is Portland cement concrete.

In the construction of canals upon steep mountain sides cement has been used for a double reason for lining the canals. By the use of lined canals a higher velocity can be attained at a given grade and the high velocities with such lining are not destructive (Fig. 3). By this means a large amount of rock excavation is saved. It is in general true that the actual expenditure for conduit through a rock cut is less with a lined than with an unlined channel.

Where a side hill construction is not through rock, but is in clay or other varieties of earth (Fig. 4), it often becomes

FIG. 4.—TRUCKEE-CARSON PROJECT, NEVADA. PLACING OF CONCRETE LINING IN POWER PLANT FOREBAY, LAHONTAN DAM.

economical or necessary to use concrete lining for the purpose of preventing seepage. The danger of such seepage is often the most serious menace to the safety of the canal by softening the earthy materials and creating veins of lubrication by which, with very slight notice, the lower banks of the canal will slide down hill and turn loose an enormous volume of water under a high head, which immediately begins its work of destruction, and in a few hours can cause damage that would pay for miles of cement lining.

In some cases where a side hill is so steep as to be at nearly the angle of repose of the material it is manifestly impracticable to maintain a canal cut into such a hill without carrying the side cut clear to the top of the slope, and in order to be safe this must

be flatter than the angle of repose, which is always an uncertain quantity. Where this work is so heavy as to be impracticable, a flume or tunnel is necessary, and the Reclamation Service has had occasion to employ concrete in the construction of both.

In the Tieton Canyon near North Yakima, about seven miles of such concrete flume was built on the mountain side upon a bench made in the steep slope (Fig. 5). The location was so difficult of access that it was found advisable to manufacture this flume in sections of two feet each at selected points in the bottom of the canyon where water, sand, gravel and transportation

FIG. 5.—YAKIMA PROJECT, WASHINGTON. MAIN TIETON CANAL, TIETON CANYON.

facilities were at hand. After manufacture these shapes were hoisted to position (Fig. 6) and the joints were sealed with cement mortar. This flume has been giving excellent service for two years.

In some regions the open soil causes heavy losses by seepage through unlined canals. This seepage, under ordinary, fair conditions, will range from one to two cubic feet per day per square foot of wetted area. In other words, the velocity of the water percolating through the soil is from one to two feet per day. In the tighter soils this may be considerably less and in open, sandy or gravelly material, may run to ten times this velocity.

It is therefore often a very important matter to prevent such losses, not only on account of the value of the water itself, but on account of damage done to lands below by excessive percolation. Various experiments have been carried out to determine the most feasible method of checking such percolation, such as puddling the canal, sprinkling with oil and lining with mortar or concrete. No lining within feasible cost has yet been developed except by the use of cement, and the Reclamation Service has lined hundreds of miles of canals with cement mortar or concrete. By this means the velocity of the percolating water can usually be reduced to a negligible quantity, ranging from one-tenth to two-tenths

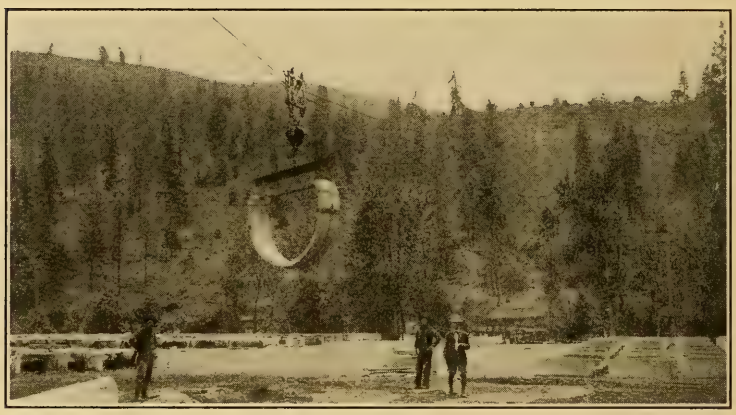


FIG. 6.—YAKIMA PROJECT, WASHINGTON. MANUFACTURE OF REINFORCED CONCRETE LINING FOR TIETON CANAL. TRANSPORTING SHAPES BY CABLEWAY.

feet per day. The experience of the Reclamation Service has demonstrated that with the increasing value of water and cost of development, the use of cement for this purpose must be relatively much greater in the future than in the past.

A comparatively recent development which has been extensively used by the Reclamation Service is the manufacture of pipe for use under pressure and for conserving the water supply. A large number of pressure pipes have been used for crossing depressions and also to serve as drops to carry water to lower elevations without destructive erosion. One pressure pipe 46 in. in diameter and over 10,000 ft. in length has been in use on the Uma-

tilla Project about two years. The major portion of this pipe is under 110 ft. static head and no perceptible leakage has occurred. Similar results with lesser heads have been obtained in numerous instances and in no case has concrete pipe used by the Reclamation Service proved unsatisfactory. In the earlier experiments it was deemed best to build the pipe continuously so as to avoid

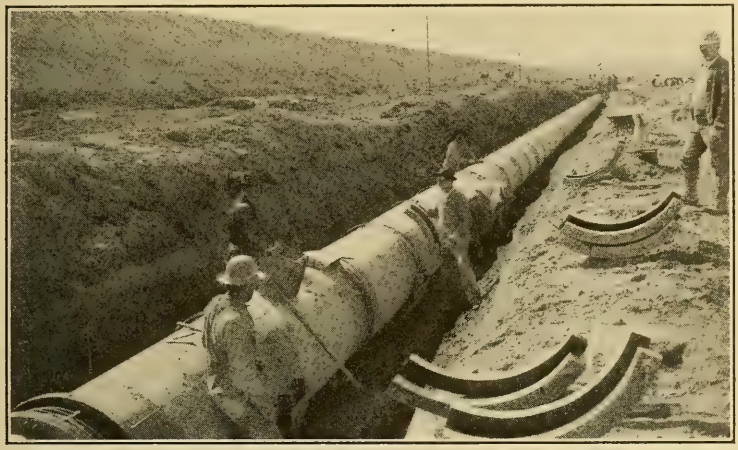


FIG. 7.—BOISE PROJECT, IDAHO. CONSTRUCTION OF CONCRETE COLLARS JOINING SECTIONS OF FOREST PIPE LINE.

joints, but of late years the major portion of pipe used, especially in the smaller diameters, has been manufactured in lengths of from three to eight feet at some yard established where materials are convenient, and hauled to the point of use and laid in trenches. The joints are sealed with cement mortar and very satisfactory results are thus obtained (Fig. 7).

THE USE OF CONCRETE BY THE BUREAU OF YARDS AND DOCKS, U. S. NAVY.

By H. R. Stanford.*

While the subject allows a wide latitude in its treatment, the author will confine his remarks mainly to pointing out the important part which concrete and cement have played in the building of the dry docks of the Navy.

Looking back to the early period of our country's naval history, the conditions will be touched upon that gave rise to the construction of the first naval dry docks.

The Navy from the beginning required home ports and fitting-out stations from which the vessels of the fleet could be provisioned, their crews assembled and the ships themselves repaired and fitted out. None but wooden vessels stoutly framed and strongly planked were then in existence, but with their increase in size it soon became evident that this country needed some better means of repairing ships' bottoms than was possible by the ancient method of beaching vessels on a soft bank at high tide and "heaving down." In 1825 Col. Laommi Baldwin, an eminent civil engineer of his day, was selected by the Government to visit England in order to familiarize himself with the features of its graving docks, or, as they are usually termed in this country, dry docks. As a result of his report, appropriations were obtained for building the first dry docks in the United States, one at the Norfolk and one at the Boston Navy Yard. Col. Baldwin was appointed chief engineer of the docks, March 28, 1827. Their construction may be said to involve the first use of cement materials in a work of importance in the Navy, the granite masonry being laid in a Rosendale or natural cement mortar. Some 29,000 bbl. of cement were used. In view of the questions that have been raised as to the durability of cement concrete exposed to the action of sea water, it is pertinent to note that these early dry docks indicate no marked deterioration of the natural cement mortar during all these years. They are in use

* Rear Admiral Corps of Civil Engineers, U. S. Navy, Chief of Bureau Yards and Docks.

today for small vessels—in fact, they are particularly well adapted for the docking of torpedo boats and submarines.

The use of concrete as the principal material in dock construction appears to date from the time of the building of dry dock No. 1, at the Navy Yard, Mare Island, prior to which the floating dock had found favor for a period. In 1872 Mr. Calvin Brown, a civil engineer in the Navy, was sent abroad and visited various public works in England and France, and on his return commented favorably, even enthusiastically, on the great utility of concrete, noting especially the French structures made of

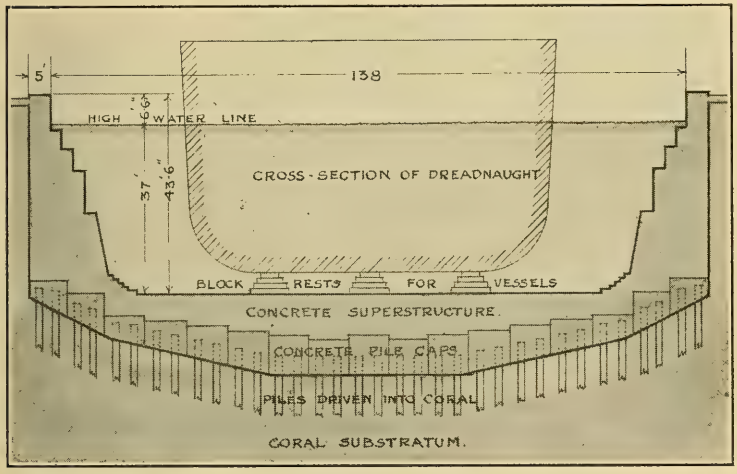


FIG. 1.—PEARL HARBOR DRY DOCK.

beton. His investigations led to the construction of the first concrete-and-granite-lined dry dock at the Mare Island Navy Yard, which was attended, however, with many physical difficulties. A period now followed during which several timber dry docks of the Simpson type were successfully constructed, but the second timber dock, known as No. 3 at the New York Navy Yard, gave so much trouble that it undoubtedly had a strong influence in again converting the naval authorities to the construction of masonry dry docks. The first Simpson dry dock at this yard was in use only a few years when it was decided to reconstruct it entirely of concrete, without any granite lining. The concrete

in this dock, which is only about ten years old, has deteriorated in places very seriously and it is only a question of a short time before very extensive repairs will have to be undertaken.

In 1898 appropriations were made providing four additional timber dry docks at other navy yards. All of these docks, however, were finally built of concrete, lined with granite masonry, mainly through the able efforts of Rear Admiral M. T. Endicott, Civil Engineer U. S. Navy, at that time Chief of the Bureau of Yards and Docks, who prevailed upon Congress as to the wisdom of such a change.

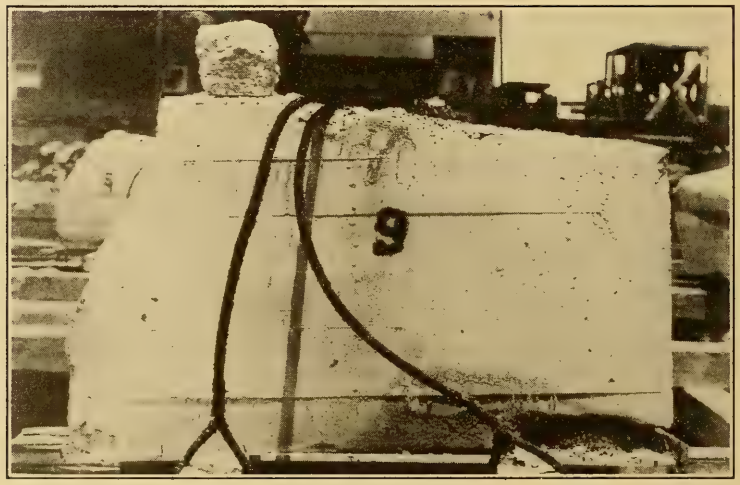


FIG. 2.—TEST BLOCK CAST UNDER SEAWATER AT A DEPTH OF 55 FT. NAVAL STATION, PEARL HARBOR, H. T.

Since the Spanish War many of the navy yards have been practically rebuilt and concrete has found a wide application in this work. For structures such as docks, sea-walls and piers exposed to the action of sea water, mass concrete built within coffer-dams and laid in the dry has more generally been used.

For structures entirely ashore, such as shops, storehouses, magazines and hospitals, reinforced concrete has been used extensively for the interior floor constructions and more recently for entire buildings. At New York, Norfolk, Charleston, Puget Sound and Olongapo, reinforced concrete has been used also of

recent years for important sea-wall and pier construction in sea water.

At the Naval Station, Pearl Harbor, there is now building the largest dry dock belonging to the Government. It will have a usable length of 1,000 ft. and will be capable of docking any vessel that can pass through the locks of the Panama Canal. The site of the dock is in a water-bearing coral rock formation made up of lenticular layers of coral rock, lava mud and coral sand, alternating and overlapping each other irregularly. A crib type of coffer-dam is being used, the material being Douglas fir. The foundation is piled and it is necessary to seal the bottom

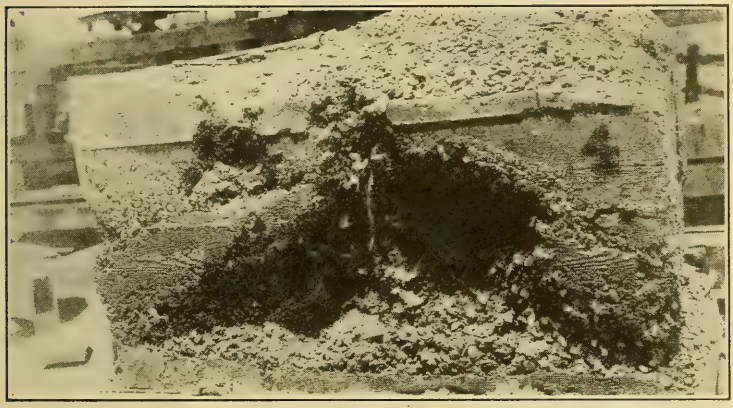


FIG. 3.—TEST BLOCK, PEARL HARBOR, H. T.

with a blanket of concrete surrounding the pile heads of sufficient thickness to resist the hydrostatic pressure when the crib is pumped out.

To deposit the concrete under water at depths from 40 to 55 ft., 18 in. diameter tremies are employed, and to obtain dense and strong concrete the bottom is divided by rectangular forms into pockets from 8 to 10 ft. square and 8 ft. high.

Extensive experiments have been carried out by filling compartments and boxes under water, using varying proportions of Portland cement, lava rock screenings, crusher dust and broken lava rock, varying the size of the aggregate and varying the sand content. With the assistance of the College of Hawaii, numerous

laboratory tests were conducted at the same time and cubes cut from the large test blocks were tested in compression. As a result of these tests it was determined that the concrete to be placed under water by the tremie method should be mixed in the proportion of one part cement, two parts sand and three and one-half parts broken stone; and the sand itself should be two parts crushed lava and one part silicious sand. The use of silicious sand is necessary to obtain a smooth and continuous flow of concrete with the least exposure to the washing and chemical action of the sea water. With this mixture the mouth of the tremie is kept buried in the mass about three feet.

FIG. 4.—TEST BLOCK, PEARL HARBOR, H. T.

Figure 1 shows a cross-section of the dock. The lower and darker masses indicate the concrete being deposited under water.

A test block 4 x 4 x 7 ft. was poured through a 15-in. tremie at a depth of 55 ft. in the sea water, using a mixture of 1 : 2 : 3.5. The broken stone content was made up of 3 parts lava rock $\frac{1}{2}$ in. to 1 in. in size, 2 parts lava rock $\frac{1}{4}$ in. to $\frac{1}{2}$ in. in size, sand-lava from crusher.

After 38 days under water the block was raised and 6-in. cubes cut from it tested in compression, failed at a load of 860 lb. per sq. in. Laboratory samples of the same mixture after 30 days failed at 1,355 lb. (Fig. 2).

A test block of 1 : 2 : 3.5 mixture. The stone ranged from

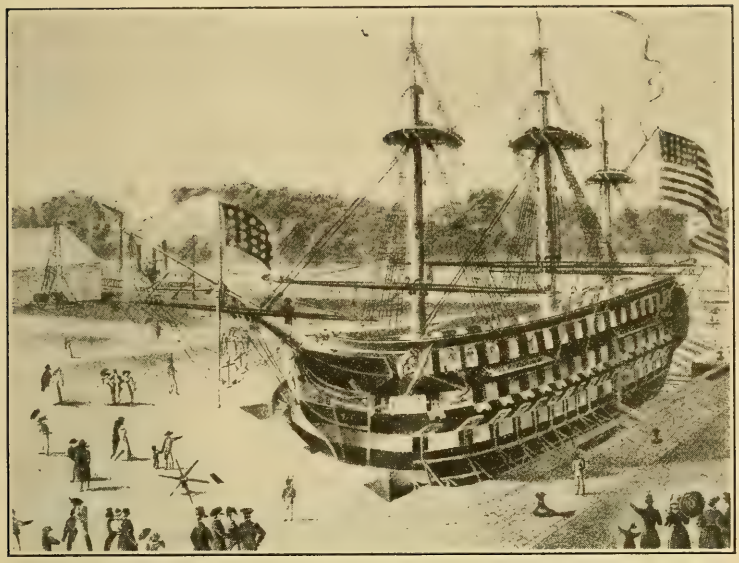


FIG. 5.—OPENING NORFOLK DOCK NO. 1, JUNE 17, 1833.
(From an old lithograph, republished by courtesy "Army and Navy Register.")

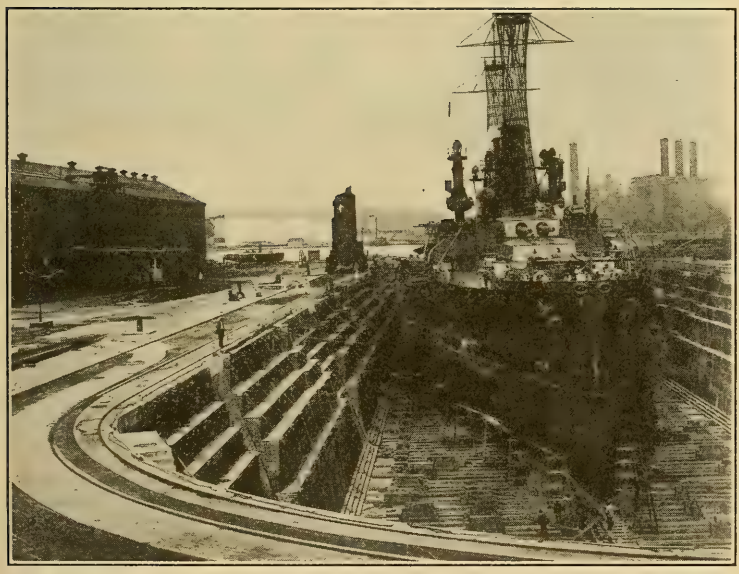


FIG. 6.—"NORTH DAKOTA" IN DRY DOCK NO. 4, NEW YORK NAVY YARD.

1 in. to $\frac{1}{4}$ in. The sand was proportioned, 2 parts crusher sand and 1 part Puget Sound sand.

Compression tests on cubes cut from this concrete (Figs. 3 and 4) after 30 days showed an ultimate strength of 2,075 lb. per sq. in.; laboratory samples showed practically equal strength for the same period. This mixture is the one now being used for the under-water concrete.

TABLE I.—RESULTS OF TESTS OF CONCRETE TEST BLOCKS, NAVAL STATION, PEARL HARBOR, H. T.

Block No.	Proportions.	Rock.	Sand.	Laboratory Sample.		Test Block.	
				Ultimate Crushing Strength, lb. per sq. in.	Age.	Ultimate Crushing Strength, lb. per sq. in.	Age.
1	1–3–5	3 parts 1½ to ¾ in. 2 parts ¾ to ¼ in.	2 parts Crusher..... 1 part Waianae.....	810	1 mo.	.....	.....
2	1–2–3½	3 parts 1 to ½ in.. 2 parts ½ to ¼ in..	Crusher	1,355	1 mo.	860	38 days
3	1–2–3½	3 parts 1 to ½ in.. 2 parts ½ to ¼ in..	2 parts Crusher..... 1 part Waianae.....	1,040	1 mo.	1,010	38 days
4	1–3–5	1½ to ¼ in.....	Crusher..... ⅜ in. to dust.....	1,580	2 mo.	.....	.....
5	1–2–3½	1 to ¼ in.....	Waianae	1,110	1 mo.	1,180	1 mo.
6	1–2–3½	1 to ¼ in.....	2 parts Puget Sound.. 3 parts Waianae.....	1,090	1 mo. (ABN.)	1,520	1 mo.
7	1–2–3½	1 to ¼ in.....	1 part Crusher..... 2 parts Waianae.....	1,180	1 mo. Approx.	1,765	1 mo.
8	1–3–5	3 parts 1½ to ¾ in. 2 parts ¾ to ¼ in..	2 parts Crusher..... 1 part Waianae.....	.....	.....	.....	.....
9	1–2–3½	1 to ¼ in.....	2 parts Crusher..... 1 part Puget Sound..	2,000	1 mo.	2,075	1 mo.

NOTE.—Concrete test blocks were cast in wooden forms varying in size from 4 x 4 x 7 ft. to 5 x 5 x 7 ft. and were poured by use of 15 in. and 18 in. diam. tremies at a depth of 55 ft. in the sea water.

The composition of the various test blocks and the results obtained are shown in Table I. The rock is obtained by crushing volcanic lava. The Waianae sand is a local coral sand obtained from the beach.

Typical examples of dry dock work and a few other structures in which concrete has been employed to some considerable extent will now be given.

Figure 5 is reproduced from an old lithograph of the first dry

dock at Norfolk, which was previously mentioned in describing the early history of naval docks, and shows the U. S. Frigate "Delaware," a wooden man-of-war in the dock. This was the first ship ever docked in a government dry dock and the event took place on June 17, 1833, the anniversary of the battle of Bunker Hill.

Figure 6 is a view of the "North Dakota" in dry dock No. 4, and is the largest dry dock at New York which has been completed within the year. On account of the difficulty met with in trying to excavate and pump out the water from an open type coffer-dam, resort was had to the construction of a continuous curtain foundation wall of concrete placed by the pneumatic

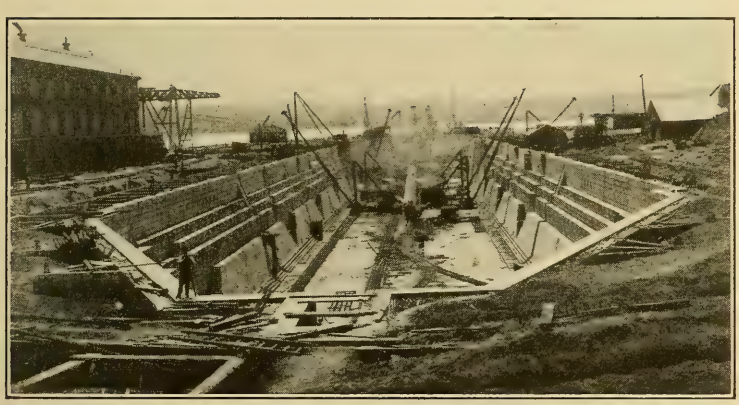


FIG. 7.—VIEW FROM HEAD OF DRY DOCK NO. 2, PUGET SOUND NAVY YARD, JUNE 3, 1912.

caisson method to carry the side walls and with piers to support and anchor the floor. The floor slab is about 8 ft. thick and is reinforced with steel. The floor is of concrete with granolithic finish. It was originally intended to line the sides with granite, but for lack of funds vitrified brick was substituted, with the sole purpose of preventing the deterioration and destruction of the concrete surfaces experienced in the case of dry dock No. 2. In this dock there were used over 100,000 bbl. of Portland cement.

This illustrates the general features of construction involved where the more usual and fairly favorable foundation conditions are experienced although the entire area required piling to support the loads.

Dry dock No. 3 of the Norfolk Navy Yard, of concrete, granite faced, has been lengthened 180 ft., making it now capable of docking a vessel 712 ft. long, 101 ft. beam, and drawing 31 ft.

FIG. 8.—PIER 8, NAVY YARD, PUGET SOUND; 3-FT. CYLINDERS BEING SET ON PIER CONSTRUCTION

of water, making due allowance for working clearances. The lengthening of the dock was accomplished within the contract time of fifteen months.

FIG. 9.—GENERAL STOREHOUSE, PUGET SOUND NAVY YARD.

Dry dock No. 2, Puget Sound, Washington, shown in Fig. 7, the largest Government dock within the continental limits of the United States, is 827 ft. long, 145 ft. wide and 47 ft. in depth.

Pier No. 8 of the Puget Sound Navy Yard is a very interesting application of reinforced concrete. Cylinders of reinforced concrete 3 ft. in diameter are molded on shore and when sufficiently hardened are placed in position, sealed, pumped out and filled (Fig. 8). The deck consists of reinforced concrete beams and slabs filling each panel and tieing the deck and cylinder reinforcing. Sway bracing of steel rods extend to base of cylinders.

Figure 9 is a view of the general storehouse, Puget Sound, entirely of reinforced concrete construction, costing $200,000.

FIG. 10.—BOSTON NAVY YARD PIER NO. 1, UNDER REPAIRS.

In the modern hospitals for the Navy all floor construction is of reinforced concrete.

A good example of the action of sea water on concrete under certain conditions is illustrated in Fig. 10, showing the condition of Pier No. 1 at the Boston Navy Yard on June 3, 1911, after a period of about only nine years' service. In the course of cutting away the old concrete to make repairs, a layer of a white putty-like substance was found which seemed to have considerable chemical activity for some time after being taken out. The

workmen complained that it burned their hands. The method of repairing the pier was to replace the face of the arch with a reinforced concrete beam.

Another important feature of the Pearl Harbor dock is the steel caisson which is to form the gate for the completed dock. The caisson was built at the Union Iron Works, San Francisco, Cal., ballasted with about 700 tons of concrete, launched and successfully towed 2,200 miles to the Hawaiian Islands.

In passing from the earliest dry docks to those of the present day we have observed that floating docks and timber docks have come and gone, whereas the early masonry docks built by Col. Baldwin are with us today, not as relics of the past, encumbering the earth, but as useful features of the naval establishment.

We trust that the concrete dry docks built by the present generation of officers of the civil engineer corps will still form a valuable function as a part of our country's public works one hundred years from now, so that posterity may hold these structures in as high regard as we now hold the historic works of our first engineer, Col. Baldwin.

CONCRETE IN RAILROAD WORK.

By M. A. Long.*

While concrete seems a new material to this generation history tells us that it is a very old one; in Italy the Roman baths and concrete bridges built 1,500 years ago are still used.

For many years the secret of making concrete appeared to be lost; at the end of the eighteenth century its use was revived and today it is used probably to a greater extent than any other building material; it is sometimes considered an infallible substitute for every other material; but to be durable it must be properly used and of good quality.

The railroads, to my mind the master builders, are using annually millions of barrels of cement, and each year records an increasing number of structures of concrete.

Concrete, properly used, is perhaps the most permanent building material we have. Since the railroads, really in their infancy, are developing so rapidly that frequent changes are necessary, they hesitate to build some structures of concrete, because of this permanency.

Compared to structures built 1,500 years ago, many railroad structures are very short lived; for example, the first railroad station in Chicago was built near the junction of the present Canal and Kinzie Streets, in the fall of 1848; that was only sixty-four years ago, and doubtless it would be in use today had progress not necessitated its replacement by larger structures, better adapted to the handling of the increased business; even in twelve years the depth of our engine houses have increased from 75 ft. to 95 ft., and we are now making plans for a house 115 ft. deep. So, in building railroad structures of concrete, though we feel they should last even longer than the sixty-four years referred to, who can say that they can be operated economically even twenty-five years hence? Therefore, were it not for the demands of progress, the speaker believes that the majority of all buildings would be built of concrete; but the tearing down of

* Architect, Baltimore and Ohio Railroad, Baltimore, Md.

concrete is difficult and, because the salvage cannot be re-used to advantage, is expensive and wasteful.

For foundation work, concrete, plain or reinforced, is almost always used, because for work of this character it is cheaper, stronger, more flexible and a shorter time is required for its construction than for any other material.

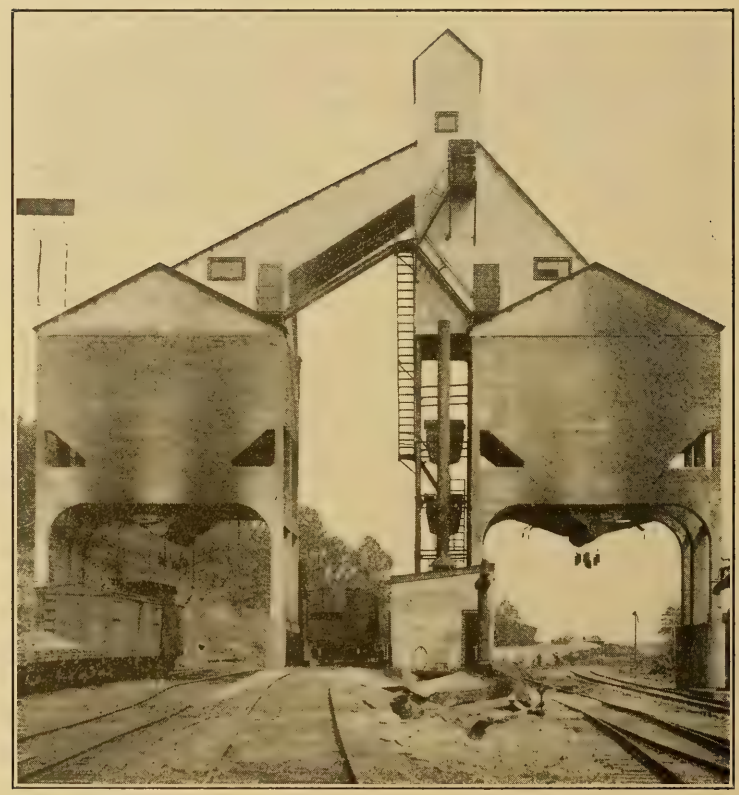


FIG. 1.—REINFORCED CONCRETE LOCOMOTIVE COALING STATION, WARWICK, O.

The Baltimore and Ohio Railroad has used it for coaling stations (Fig. 1), where 800 tons of coal is stored over its main tracks, and it has proved very satisfactory.

It has also used concrete in the construction of water (Fig. 2) of large capacities and under pressures. Three tanks of 100,000 gallons capacity have been built, with one of

which there has been considerable trouble. This tank, 24 ft. in diameter and 80 ft. high, not only furnishes water for the engines but also for fire protection. The water line in the storage

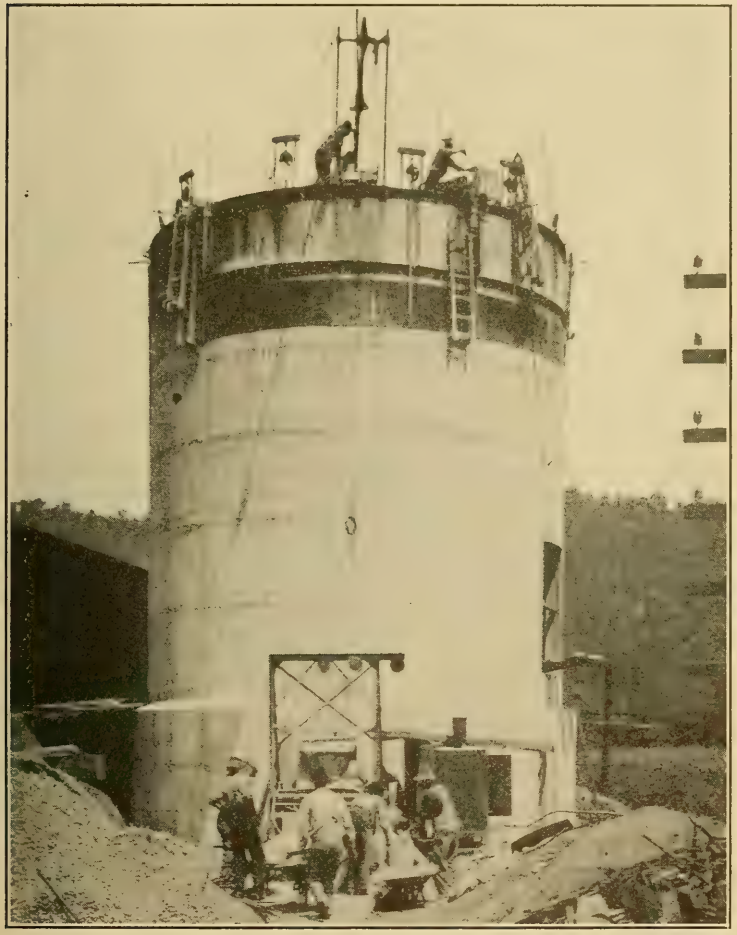


FIG. 2.—REINFORCED CONCRETE WATER TANK UNDER CONSTRUCTION, SIR JOHN'S RUN, W. VA.

compartment is 50 ft. from the base of rail. The tank was finished late in December, and was filled to within 4 ft. of the top a few days later; on the day it was filled the temperature

fell to zero. It so happened that no water was taken from this tank until several days after it was filled. In the meantime, a layer of ice about 18 in. thick at the outside and not more than 1 in. thick at the center, had formed. About the time the tank was emptied the weather moderated, the ice thawed and fell and in breaking loose tore off the waterproofing; this allowed the water to percolate through the walls and freeze on the outside in masses weighing a ton or more; when the weather again moderated, this ice thawed and fell off; it was found that the walls were damaged very little, and with an application of waterproofing on the inside and a coating applied to the exterior with a cement gun, the tank held water satisfactorily, and there has been no trouble with it since.

From the experience thus gained, it is recommended that concrete structures of this character be coated inside and outside with waterproofing, and be allowed to cure before being filled with water.

The trouble noted was no doubt caused by failure to protect the concrete from freezing until it had thoroughly hardened; the moisture in the concrete froze, and in thawing caused leakage.

The two other tanks are 53 ft. high, and two reinforced concrete reservoirs built on the ground 55 ft. in diameter and 12 ft. high, each holding 200,000 gallons, are giving satisfactory service. The latter are coated inside and out with waterproofing compound.

Reinforced concrete cinder pits have been built, into which hot cinders are dumped and then drenched with cold water. To meet these conditions the concrete must be faced with vitrified brick, as alternate heat and cold cause spalling. Slag has been tried as a substitute for stone in the concrete for these pits, and while it is an improvement, it is not altogether satisfactory. Trap rock is the most satisfactory material for concrete in cinder pits.

Pump houses, in which the pumps are placed in a watertight compartment below the outside water level, have also been successfully built of concrete.

Subways under the railroad tracks connecting one platform with another are also built of concrete. The one the speaker has in mind is below the water level, and it was necessary to waterproof it. During construction the water was kept down by pumping and a cinder concrete base was laid to receive the mem-

brane waterproofing. After completion it was found that there was a greater head of water than was figured and some leaks developed. In repairing it was found that the concrete, made of soft coal cinders, had not set up; it has since been learned that others have had similar experience; upon investigation it was found that the soft coal cinders contained a great deal of sulphur. This is not true of hard coal cinders, and they are the only kind that should be used in concrete.

One reinforced concrete engine house has been constructed, of which we are reasonably sure that it will be permanent; this has a wooden roof since it was felt that the escaping steam would condense on a concrete ceiling and drip on the engine jackets.

There was considerable trouble at times from dampness during hard, steady rains, in an elevator for the storage of grain, constructed of reinforced concrete, although the concrete was of a rather wet mixture; waterproofing was applied to the outside of the walls and since that time there has been no trouble from dampness.

The Baltimore and Ohio Railroad is erecting an eight-story reinforced concrete warehouse in New York City, in which no waterproofing will be used in the exterior walls above the ground line, for the reason that the concrete, by careful mixing, placing and working into place, can be made sufficiently waterproof.

Jointly with the Lake Shore and Michigan Southern Railway, the Baltimore and Ohio Railroad built a passenger station at Gary, Ind., which is practically all concrete; the building (Fig. 3) is classic in design and the lines and shapes are the same as if cut stone had been used.

During the past year it was found that watch boxes and telephone booths of reinforced concrete could be bought more cheaply than they could be built of wood, and those of concrete have been adopted as the standard.

Plans for some small shop buildings, within the past year, called for wood construction, also for steel frame covered with metal lath, plastered on both sides with cement mortar, making a concrete wall approximately 2 in. thick; since the bids for the steel frame and concrete were as low as those for wood construction, concrete was adopted.

In our track elevation work at Chicago, concrete is the principal material used, and the results obtained are very satisfactory; the design and general appearance of the work through the parks has been very favorably commented upon by the Park Board (Fig. 4).

There are many reasons why concrete is used by the railroads in preference to other materials, some of them being:

1. Its rigidity, especially in buildings containing machinery. The vibration in a properly constructed concrete building is negligible.

FIG. 3.—REINFORCED CONCRETE RAILROAD STATION, GARY, IND.

2. Its low cost of maintenance. With a well-constructed concrete structure, the maintenance should be very low, and the structural parts should last indefinitely.

3. Its fire resistiveness. Reinforced concrete structures have so high a resistance to fire that it is only necessary to carry insurance on the contents.

4. Its waterproofness. Concrete can be made practically water-tight against small pressures; to obtain this result it should be mixed wet and well worked into place.

5. Perhaps the most important item is the cost. Compared

to a structural steel fireproof building, its cost is lower, but it is more expensive than ordinary mill construction; a flexible form that can be used over and over again and flexible enough to fit any wall or column would materially reduce the cost.

There are a number of other things that you who are making a life work of concrete in its various phases should consider.

Concrete would be universally used as a wearing surface for freight house floors, passenger platforms, etc., were it not for the fact that its surface is likely to become chipped and broken, and then to wear very fast. We are now making some tests with

FIG. 4.—REINFORCED CONCRETE BRIDGE, TRACK ELEVATION WORK, CHICAGO, ILL.

ironite worked into a rich mixture for wearing surface, and while this increases the wearing qualities, it is not entirely satisfactory. Concrete floor surfaces dust badly, and it is necessary to resort to the use of linseed oil and shellac, or to hot silicate of soda, which eliminates the dust for a time.

As concrete cures it has a tendency to crack, and this gives it the appearance of having veins, especially where moisture has had a chance to enter these small cracks. Where the mass of concrete is waterproofed this cracking is serious and these incipient cracks ruin the appearance of the surface; where cracks occur moisture enters which results in disfiguring surface efflorescence;

if this problem were solved the speaker is sure that many more buildings would be constructed with a concrete wearing surface. Many of these cracks are no doubt the result of using a very rich mixture; when it is necessary to patch the surface, the mixture used for the patch should be no richer than the mass.

Another step forward in the use of concrete would be to give it color, not by painting the exposed surface, but through the addition of coloring matter, mixed or probably ground with the cement; great care must be taken to properly proportion each batch. This is only a suggestion and the necessity for avoiding any chemical action of the coloring material on the concrete may make this a hard problem to solve. The speaker has tried coloring the concrete in two different structures; in one a sand of a reddish-brown tinge, which gave a rich, warm buff color, was used; in the other a similar sand, but of a lighter color was used and the concrete had a color a little darker than cream. The speaker is quite sure that concrete will have many more friends, especially for use in residences, if it can be given a warmer color than its natural one.

Concrete is so lasting that you must be doubly sure to get a pleasing design and, in my opinion, your success lies in handling it in large surfaces. Do not try for too much detail, such as you see in some structures built of cement block.

These remarks are based on practical experience, and the speaker trusts they will stimulate discussion of this interesting material.

APPLICATION OF CONCRETE IN THE ABOLISHMENT OF GRADE CROSSINGS IN THE CITY OF PHILADELPHIA.

By James W. Phillips.*

Historical.

A brief review of the history of the abolishment of grade crossings in the City of Philadelphia may be of interest. Prior to the consolidation in 1854 the area of the city was 2.75 sq. miles; it now has an area of 129.5 sq. miles, which includes what was formerly the County of Philadelphia, embracing twenty-seven outlying districts; each of these districts had much unoccupied territory and was governed by its own local authorities.

From the advent of the railroads, about 1832 up to the time of the consolidation, the local authorities granted the railroads the privilege of occupying highways and streets, for few ever expected that this territory would be built up as closely as the residential and business centers of the city was at that time.

As a result of these circumstances and of the increase in the population and area of the city, there are today about 794 railroad street crossings, of which 413 are at grade, and 223 are over and 158 are under the railroads. Active steps are being taken to abolish all grade crossings.

The number of grade crossings is not large when compared with the number in other cities. For example, in Chicago there are about 3,000 railroad street crossings, of which 1,750 are at grade; the City of Denver, with a much smaller population than Philadelphia, has 832 railroad street crossings, of which 817 are at grade.

In 91 cities, each with a population of at least 30,000, having a combined population of 23,000,000 the total number of railroad street crossings is 17,850, of which 13,700 are at grade.

With these figures before us, it is pertinent to ask what Philadelphia has done and is doing toward the abolishment of

* Assistant Engineer in Charge, Department of Public Works, Bureau of Surveys, Philadelphia, Pa.

grade crossings. The first important work of this character was the removal of four grade crossings on the line of the Connecting Railroad and the establishment of 14 additional crossings under this railroad.

The next and far more important work was the construction of the Pennsylvania Avenue Subway and Tunnel by which 16 grade crossings were abolished along the line of the Philadelphia and Reading Railway at a cost of $5,500,000.

Then followed the elevation of tracks of the Philadelphia and Trenton Railroad on Trenton Avenue, abolishing 30 grade crossings.

The latest and most important work now nearing completion is the elevation of the tracks of the Philadelphia, Germantown and Norristown Railroad between Green Street and Wayne Junction, a distance of 3.25 miles through a densely built-up portion of the city, by which 28 grade crossings are eliminated and two additional streets opened up.

It is the purpose of the speaker to describe this latter in some detail and thus illustrate the part played by concrete in the important work of eliminating grade crossings.

OUTLINE OF THE WORK.

The Philadelphia, Germantown and Norristown Railroad, constructed in 1832–33, was the first railroad built into Philadelphia, if not in the United States; in 1834 trains were operated between Germantown and its terminus at Ninth and Green Streets.

Commencing at Green Street, the four-track railroad is carried on an earth fill and concrete retaining walls to the south side of Brown Street, bridging Fairmount Avenue; from the south side of Brown Street to the north side of Jefferson Street, it is carried on a steel-trough-floor viaduct; from the north side of Jefferson Street to Broad Street by earth fill and concrete retaining walls, bridging streets and coming to the grade of the present tracks at about the south line of Broad Street. It begins to rise again at Seventeenth Street and Indiana Avenue, from which point it is carried on an earth fill between concrete retaining walls until it meets the grade of the present track just north of Hunting Park Avenue (Fig. 1).

All bridges are of the plate girder type, with two exceptions, at Berks and Norris streets, where there was sufficient height to permit the construction of concrete arches.

LOCOMOTIVE COALING STATION.

An interesting piece of construction is the locomotive coaling station, located in the East Green engine yard (Fig. 2). The building is of reinforced concrete throughout and is 46 ft. wide and 101 ft. long; it rests on a reinforced concrete foundation; the columns and girders supporting the building are of steel enclosed in concrete. The structure covers seven tracks, six for

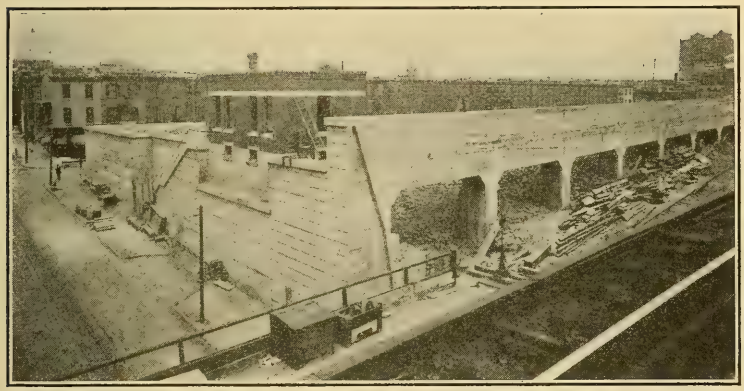


FIG. 1.—OFFICE, AND REINFORCED CONCRETE RETAINING WALLS, NORTHEAST CORNER TENTH AND BERKS STREETS.

coaling and dumping ashes and one for removing ashes; the latter is located at the west side of the building, adjacent to the main tracks. The coal is delivered to the station from a track located at the west end of the building at the same elevation as the main tracks, into hoppers, from which it is elevated by bucket conveyors to the pockets; these are 12 in number (two over each coaling track) and have a combined storage capacity of 2,000 tons; each pocket is provided with steam pipes for thawing frozen coal. The ash pits, one under each coaling track, are so arranged as to permit the locomotive to receive coal and dump its ashes at the same time; each pit is of concrete lined with brick and provided

with a sprinkling device for quenching the ashes. The ashes are conveyed from these pits by bucket conveyors to the ash pockets having a capacity of 250 cu. yd. and located at the west end of the building immediately over the ashes track.

Six sand bins are provided, one over each coaling track, each with a capacity of 4 cu. yd.

COAL POCKETS.

Just east of the main track and between Berks and Norris streets, two lines of double-track coal pockets have been con-

FIG. 2.—LOCOMOTIVE COALING STATION, PHILADELPHIA & READING RAILWAY, PHILADELPHIA, PA. SHOWING USE OF CONCRETE IN THREE DIFFERENT TYPES OF STRUCTURES.

structed, each about 46 ft. wide and 500 ft. long, having a combined storage capacity of 26,000 tons (Fig. 3).

The combined type of wood and concrete was used throughout, both in these and those constructed elsewhere. The construction in general consists of a series of concrete piers 25 ft. centers and 46 ft. long, having a thickness of 3.5 ft. from the floor of the pockets to the footings and 2.5 ft. above this floor. The footings are of reinforced concrete 10 ft. wide and 49 ft. long. The piers carry the steel plate girders and track stringers. The coal is dumped from the cars into the pockets, from which it is drawn by chutes into the delivery wagons; the floor of the pocket is not less than 10 ft. above the paved driveways. A longitudinal

partition divides each pocket into two. The pockets have a steel frame, corrugated-iron covered shed.

One feature that may be noted is the reinforced concrete office building constructed beneath one pocket on the Berks Street side. There were nearly 5,000 cu. yd. of concrete used in the construction and the cost complete, including paving of yard, stables for 114 horses and office building, was $273,578.00.

CONCRETE ARCHES AND RETAINING WALLS.

At both Berks Street and Norris Street there was sufficient difference between the street elevation and the railroad tracks as to permit the construction of concrete arches.

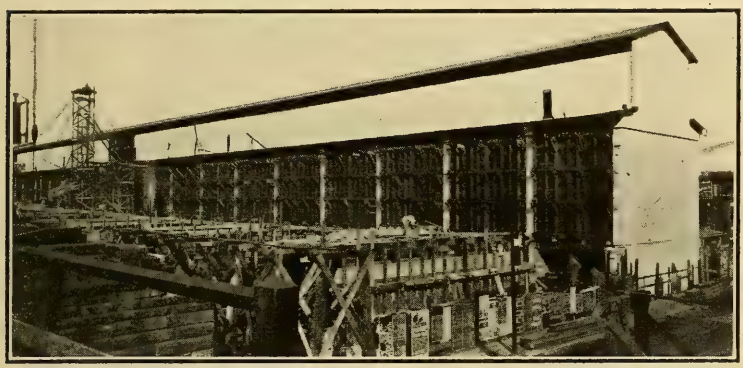


FIG. 3.—COAL POCKETS FROM SOUTHEAST CORNER BERKS STREET AND PHILADELPHIA & READING RAILWAY.

These arches have a span of 50 ft., a crown thickness of 3 ft. and a rise of about 16 ft.

For the maintenance of railroad traffic it was necessary to construct each arch in two sections, diverting this traffic as each section was completed. The two sections of the arch ring were tongued and grooved, and anchored together with $1\frac{1}{4}$ in. diameter rods 8 ft. long, one-half being embedded in the ring of each section; this was the only reinforcement used in the arches.

The Berks Street arch was waterproofed with five layers of felt laid in a compound and protected with a layer of 1 : 3 mortar 1 in. thick, at a cost of 23 cents per sq. ft.

The Norris Street arch was waterproofed with $1\frac{1}{2}$ in. of asphalt mastic, composed of one part asphalt and four parts of sand and screenings, laid in two layers of $\frac{3}{4}$ in. each, at a cost of 30 cents per sq. ft.

The concrete used in the arch rings, bridge seats and pedestal blocks was mixed in the proportions of 1 : 2 : 4 and cost $7.50 per cu. yd., while that used in the retaining walls, abutments and copings was mixed in the proportions 1 : 3 : 6 and cost $6.75 per cu. yd. for copings and $6.00 per cu. yd. for walls and abutments.

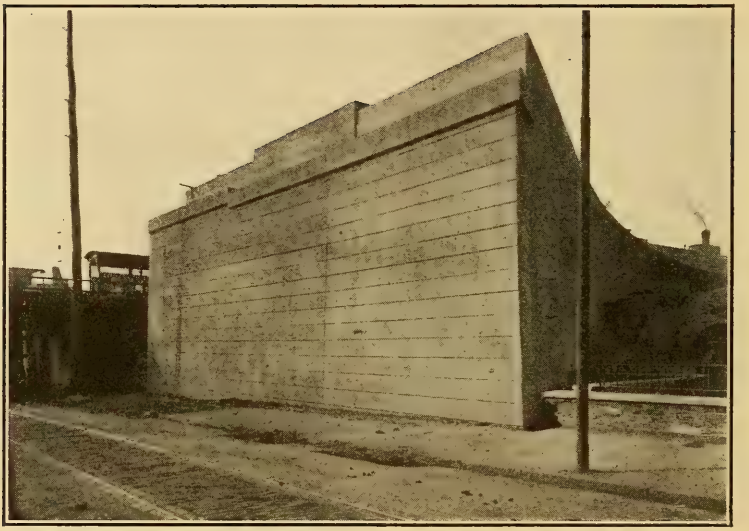


FIG. 4.—CONCRETE ABUTMENT NORTH SIDE OF NORRIS STREET EAST OF RAILROAD.

FREIGHT YARDS.

Freight yards were constructed at Eighth and Master streets, Tenth and Berks streets, and York and Cumberland streets. These were raised from the street grade level to the new elevation of the main tracks with three per cent driveways from the street level. The ramp and retaining walls in each case being constructed of concrete.

The only reinforcement used in the retaining walls shown in Fig. 1 were the rods in the bottom and side of the portion between

the piers. Expansion joints were provided over every second pier, using $\frac{1}{8}$-in. asbestos boards.

All walls were backed with dry stone, and along their bottom offset a 6-in. vitrified clay drainage pipe was laid on a one per cent grade; this pipe was carried through the abutments and walls and connected with the sewers in the streets.

All concrete abutments and retaining walls facing streets were, with one exception, given the scrubbed finish* as illustrated in Fig. 4. This finish was also used on the exposed faces of the two arches. The ramp wall along the driveway leading to the Tenth and Berks streets freight yard was peen hammered.

In all concrete walls and abutments expansion joints were provided about every 50 ft., using $\frac{1}{8}$-in. asbestos fire felt boards.

A concrete pumping post, was constructed in the York and Cumberland streets freight yard, at the terminus of a three per cent grade.

All cement was tested in accordance with the rules recommended by the Special Committee on Uniform Tests of Cements of the American Society of Civil Engineers.

UNIT PRICES OF MATERIAL.

This table gives the average unit prices of some of the important materials used in the work, except waterproofing:

	PER CU. YD
First-class masonry, concrete backing	$9.50
First-class masonry, third-class backing	8.14
Second-class masonry, third-class backing	7.42
Third-class masonry, above neat lines	5.71
Third-class masonry, in foundations	5.43
First-class arch masonry	11.25
1–2–4 concrete arch masonry	7.95
1–3–6 concrete copings	7.54
1–3–6 concrete masonry ($1\frac{1}{2}$-in. stone)	6.45
1–3–6 concrete masonry ($\frac{3}{4}$-in. stone)	6.45
1–2–4 concrete bridge seats ($\frac{3}{4}$-in. stone)	7.62
1–2–4 concrete pedestal blocks ($\frac{3}{4}$-in. stone)	7.62
1–3–6 concrete masonry, above neat lines, street face	6.74
1–3–6 concrete masonry, above neat lines	6.32
1–3–6 concrete masonry, in foundations	5.82
Dry stone back of walls and abutments	2.65

* Proc. N. A. C. U., Vol. III, p. 129.

GENERAL DATA.

Street railroad crossings abolished	28
Number of steel bridges constructed	20
Number of concrete arches constructed	2
Structural steel	27,180 tons.
Third-class masonry	67,500 cu. yd.
Concrete masonry, all classes	151,800 cu. yd.
Cement	231,800 bbl.
Earth embankment	431,000 cu. yd.
Street excavation	140,000 cu. yd.
Street paving, all kinds	134,312 cu. yd.
Sewers constructed	17,840 lin. ft.
Miles of single permanent track	30.1
Length of viaduct	3,365 lin. ft.
Total steel in viaduct	30,010,000 lb.
Averaged weight of four-track viaduct, per lin. ft.	7,617 lb.
Averaged weight per sq. ft. of viaduct	152 lb.
Area of floor of viaduct	197,300 sq. ft.

SOME RECENT APPLICATIONS OF CONCRETE IN RAILROAD WORK.

By Frederick Auryanson.*

The fact that the railroads of the United States use such a large percentage of the cement manufactured in this country makes the discussion of railroad structures before this convention quite appropriate.

The reputation of concrete for railroad structures is based on the sure foundations of:

1. *Cheapness.*—It is cheap because its constituents are cheaper and easier to handle than the units required in any of the other materials usually involved in large work. This reduces the construction plant to a minimum.

2. *Speed of Construction.*—Not only is the process of mixing and placing concrete a more rapid one than the building of stone masonry, but delays in the receipt of particular parts are avoided, the forms can ordinarily be built to suit the changing phases of the work, and little if any extra expense is involved in caring for matters of immediate importance or urgency. Changes which would postpone the completion of wood, stone or metal structures for weeks or even months, can be effected in concrete with little delay, expense or risk.

Plans may be changed at the last moment without involving any considerable delay in the execution of the work, whereas other building materials have to be ordered in advance and when the channels of production are congested and orders have to be placed long in advance, changes are almost out of the question, or are effected only at great cost either of time or money.

3. *Permanence, with Increasing Strength.*—Steel structures require a considerable annual outlay for upkeep, the neglect of which always proves to be false economy; while for most uses employing wood, its short life and increasing scarcity and value, combined with poorer quality, long ago made concrete a close

* Bridge Engineer, Long Island Railroad, Jamaica, N. Y.

competitor in railroad structures, where durability is a prime essential.

A further advantage in the use of a plastic material such as concrete is that irregular shapes are comparatively simple of construction. In a skew arch, for example, the stone cutting is not a simple problem, whereas with concrete the principal thing to take care of is the proper disposition of the reinforcement, the regular forms being merely extended beyond the skew end and provided with slight additional bracing to take up the unbalanced thrust of the wet concrete. Also, concrete can be deposited in masses as large or small as desired, depending upon the size of the job and the capacity of the mixer, and it can be either deposited in place or molded into suitable shapes beforehand and placed after it has hardened.

A recent development along this line is the use of concrete track slabs, which may be cast either in a construction yard or near the bridge site and, after attaining sufficient strength, be put into permanent place during a favorable interval between trains. This type of construction has been favored largely of late, but on the Long Island Railroad the system has not been employed, as it is considered better to use structural steel members capable of supporting the full load, usually erecting them after diverting the traffic, which is not restored until the concrete slab has fully set. Several years ago thirty days was considered the least safe time for trains to pass over bridge slabs, but last month trains were run upon a half through bridge over Main Street, Flushing, L. I., twelve days after the last concrete was deposited; moreover, the intention was to use the bridge within a week after the slab was finished, carrying the ties temporarily upon eight lines of 100-lb. rails per track to avoid cracking the slab. The steel frame is also a precaution against indiscretions of the field force, such as taking down forms too soon; moreover, the support of the forms is easier and cheaper than without the steel structure, and the objections to shoring and bracing are eliminated. Highway bridges with I-beam floor construction which span more than two tracks have heretofore been provided with intermediate supports consisting of steel columns and girders, which were encased in concrete for protection. A logical development of this system has led to the adoption of thin reinforced concrete

intermediate walls, with refuge openings if the length is over sixty feet. Three such are designed and will soon be built at a saving of $7,000 by omitting the columns and girders.

The first considerable use of concrete on the Long Island Railroad was in the Atlantic Avenue Improvement begun in 1901, whereby grade crossings for several miles through the City of Brooklyn were eliminated. The construction included two subway sections each approximately a mile long. While this work was undertaken shortly after the beginning of work on the original New York Subway, the tunnel sections differed materially. The floor of the Long Island Railroad tunnel was paved with a concrete invert designed to take the thrust at the toe of the retaining wall, but it proved, in addition, to be very valuable as a foundation for the track, which, probably on this account, has required much less maintenance than track carried by ballast on the ordinary subgrade. Perhaps, however, the credit is not all due to the concrete flooring, as the roof of the subway protects the track and its supports from the weather and from excessive variations of temperature. At any rate, because of the good condition of these subway tracks, an extensive use of concrete is contemplated in the track work of the Jamaica Improvement, now under way, the intention being to lay an 8-in. concrete slab under all switches and cross-overs. The New York Central Railroad has also planned for similar construction where their tracks cross soft ground. The suggestion is ventured that the time is probably not far distant when a concrete floor will cover railroad sub-grades on sharp curves and at all other points where main line tracks are subject to exceptionally severe service and therefore require stronger foundations than simple ballast upon earth.

The roof of the Atlantic Avenue tunnels consists of I-beams or girders spaced 5 ft. apart, with unreinforced concrete jack arches 1 ft. thick at the crown. In this first example of combined steel girders and concrete no attempt was made to encase the bottom flanges of the beams and girders, as the clearance was close and it was thought desirable to save every inch of height. Moreover, there was some question of the feasibility of depositing concrete around and holding it securely to the bottom flange. Later experience has proved, however, that these fears were

groundless and that it is by all means desirable to completely encase all steel over engine tracks.

The Brooklyn terminal at Flatbush Avenue was the final section of the Atlantic Avenue Improvement to be completed. The first intention was to cover only part of the area of the terminal, the roof to serve as a storage yard for the cars of a projected surface line. This scheme was abandoned and a surface freight yard was laid out on the roof over the depressed passenger terminal, and the steel columns and girders for this purpose were all erected when the plans were changed again, raising the freight tracks enough to give 14 ft. clearance over the street. The freight house, 120 ft. wide and 400 ft. long, could not be made to fit the location of the steel, so a concrete grillage of cellular design was built on top of the original roof steel, accomplishing easily what would have been a most tedious and expensive undertaking without it.

The use of concrete piles has not been very extensive on the Long Island Railroad until the present year, in which about 500 16-in. 28-ft. piles, some rolled and some cast on the flat, were used to support the five 1,000-ft. platforms of the Jamaica Station. As these were built on a fresh fill of about 20 to 25 ft. height, considerable settlement is feared; but the track grade being fixed, the platforms must be carried by unyielding supports, for which conditions concrete piles seemed particularly well adapted, and besides were estimated to cost less than longitudinal walls, even with generous openings, but with footings spread to reduce the bearing pressure to a safe value. As a further great advantage, the driving process compacted and settled the filling within a zone of 4 to 5 ft. radius, so there is even less probability of settlement than was anticipated (Fig. 1).

The experience with the rolled piles was not very satisfactory, as quite a large percentage broke either in handling or driving; but of 300 octagonal cast piles, not one was broken either in handling or driving and many of them were made during quite cold weather about a year ago. There are now under construction about 1,600 more of various lengths for several structures about to be built on soft ground; as their cost is not over 50 cents per foot ready to drive, they will probably be used hereafter instead of creosoted wooden piles, where the permanent saturation of the ground is more or less uncertain.

The five concrete platforms now being built at Jamaica will be each 22 ft. wide, and supported at 14-ft. intervals by bents of two of the above-mentioned concrete piles. The surface is inclined slightly from the center towards a line about 18 in. from the edge of the platform, and from this low point a rise of $\frac{3}{4}$ in. to the edge of the platform is provided to prevent passengers from slipping towards the edge. The surface is checkered in 2-ft. squares to facilitate the escape of rain water and is also indented with a spiked roller to avoid the excessive smoothness of a concrete surface, which results from the attempt by troweling to make it dense and non-absorptive. Slipperiness is particularly

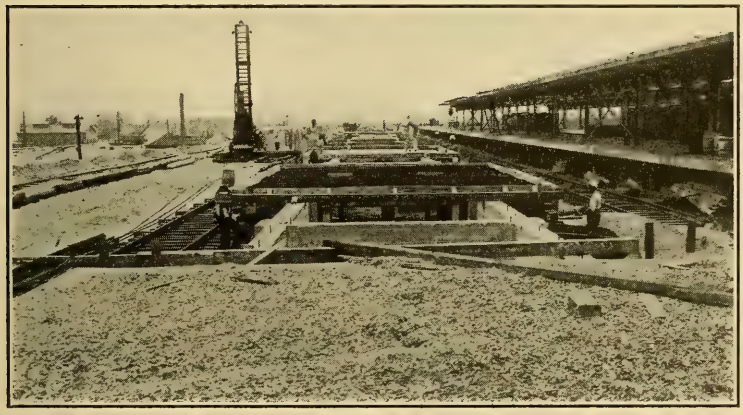


FIG. 1.—JAMAICA STATION; REINFORCED CONCRETE PLATFORM SUPPORTED ON 28-FT. CONCRETE PILES.

objectionable on stairs, and on this account, in recent stair construction for the Jamaica Improvement and elsewhere, instead of finishing the treads of concrete, bluestone of a dense, sharp, gritty texture has been used. Experience indicates that these treads will last under heavy traffic for many years, and after they have worn badly they can be turned over and used again; as their first cost is practically no more than that of the so-called "Safety Treads," probably the less durable "Safety Treads" will not again be used in our work, especially where they are exposed to the weather.

In the work of the Jamaica Improvement there is involved

one system of tracks leading eastward to the central main line and north side, and another set to the south side and the various beaches; similar conditions prevail with respect to the tracks leading westward from Jamaica, some of which turn to the northward towards Long Island City and the Pennsylvania Station in New York, while another series extends westward to the Brooklyn Terminus at Flatbush Avenue. These diverging tracks in connection with the segregation of the east-bound and west-bound traffic at Jamaica involve three "Jump" crossings, two of which are reinforced concrete boxes of double tube section and the third is a steel frame and concrete deck structure which carries four tracks over the others. In connection with the first of the reinforced concrete boxes a peculiar condition was encountered in the foundation of its east end, which came directly over Maure Avenue, a street carried under the general elevation of the new yard through a 32-ft. arch. This construction would have been very difficult with any other material than concrete, but under the circumstances an extremely satisfactory solution was developed, the wing wall for the box being designed as a concrete beam to carry both its own weight vertically and the thrust of the earth horizontally. The Maure Avenue arch (Fig. 2) was an interesting work, and though of only 32-ft. span, partly made up for this by its length, which is 510 ft. The centers consisted of steel ribs and wooden lagging upon which the reinforcing rods were assembled as frames and concrete poured in the ordinary way in sections of about 25 ft.; but the unusual length of the structure and the consequent duplication made it very cheap. Another similar arch 551 ft. long was built for Foley Avenue, and a third 211 ft. long is projected for Church Street, so that altogether the actual cost of form material for these three arches of 1,272 ft. aggregate length, becomes almost negligible.

At stations and at some other points, public convenience demands crossings for pedestrians. At grade there is added to the danger from trains, that of the third rail; the overhead bridge is usually unsightly, and people object to climbing; whereas they will go down stairs into a subway though a climb out awaits at the other end. A number of these have been built from 6 to 10 ft. wide and about 7 ft. high. Only concrete has been used during

the last six or seven years. In some cases where the ground water level was only a foot or two below the track, the difficulty of waterproofing, while great, was successfully overcome.

A problem of insufficient footings for a viaduct on filled ground is before us. What material other than concrete could solve it? The proposition is to shore up the structure, column pedestals and all, and after excavating two feet or so below, insert either beams or rails unless by means of rods a simpler solution in reinforced concrete can be found.

FIG. 2.—FRONT OF CONCRETE RETAINING WALL WITH 53 FT. CLEAR SPAN OVER MAURE AVENUE ARCH. REINFORCED TO ACT AS A BEAM HORIZONTALLY AND VERTICALLY.

Another problem is the repair of the brick floor arches of two bridges at Long Island City. The mortar pointing has been deeply eroded under the force of the blast from switching engines, whose stacks are but two or three inches clear of the structure. While a definite plan has not been adopted as yet, some experiments with the cement gun indicate that not only could the joints be repointed, but steel rods could be built into a concrete facing of gunite between the soffit of the arches and the clearance line. Experiments and calculations indicate that enough steel could be incorporated into the structure to make the floor self-

supporting, or, in other words, relieving by so much the duty now devolving upon the badly corroded I-beams which carry the arches. At the same time the flanges of the beams would be covered to prevent further deterioration.

To avoid the expense of piles, reinforced concrete wall footings have been very successfully used on Long Island Railroad work, extending them so that uniform pressures are obtained over the whole base. Generally, column pedestals are of a depth to require reinforcement for tension but not for shear, this type being estimated cheaper than gravity sections. Where the foundation is very yielding, an important factor in the design of a

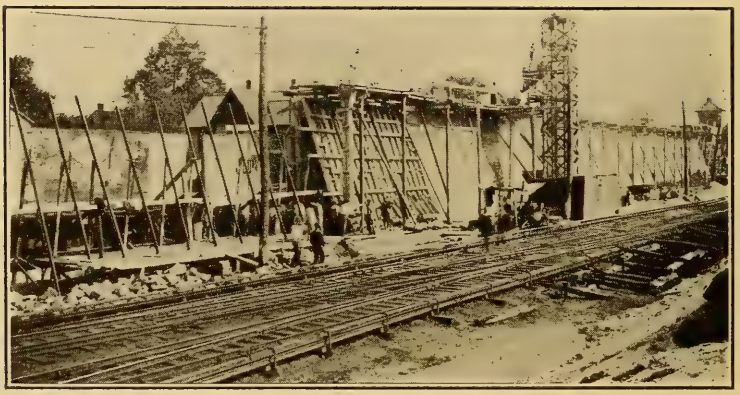


FIG. 3.—SEMI-REINFORCED CONCRETE RETAINING WALL. THICKNESS 20 PER CENT OF HEIGHT.

wall is its weight, which is a minimum with the reinforced type; the very thin walls with buttresses have but slight additional advantage over a wall of 20 per cent section, and the latter is a great deal easier to build. Most of our work has employed gravity walls, but the semi-reinforced section, using either old rails or large rods, is making a very favorable showing (Fig. 3). On fresh filling and when the precise position of the wall is not vital, the lengthening of the toe is a simple and cheap expedient to avoid settlement of a wall. But when it is important for a wall to stay where it is built, the footings may be carried to the original surface through the filling, coring out a generous section of the foundation course. In one bridge foundation on soft muck

where piles were used, the thrust of the embankments on the two abutments was balanced by concrete struts 3 x 4 ft., carried on piles just below the street surface. Before the struts were finished the filling behind one abutment was begun, but was suspended when it was found that one wing wall had begun to rotate, but this action was stopped by completing the struts.

Round-house construction offers a particularly favorable opportunity for concrete. The Long Island Railroad is fortunate, however, in the tendency to extend the electric zone, reducing the mileage operated by steam, so there has been no new round-house construction for some time. While the products of

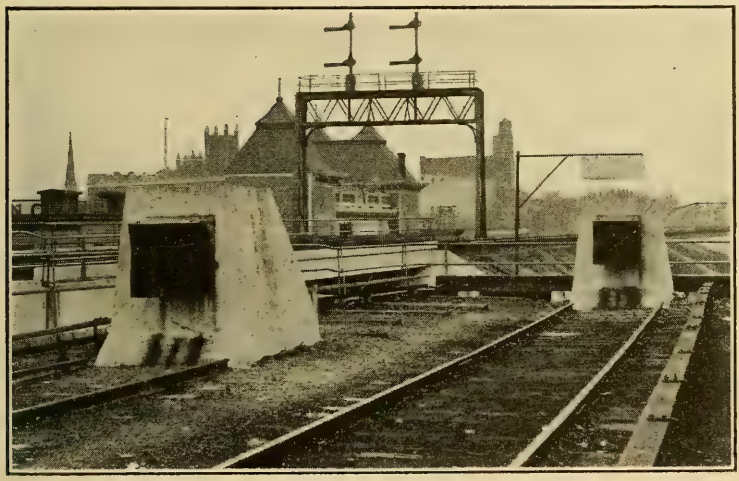


FIG. 4.—CONCRETE CAR BUMPERS AT JAMAICA, L. I.

the combustion of coal are injurious, coal itself is also, both to steel and wooden trestles, so that all new coal trestles on Long Island have either concrete piers or the steel members are encased in concrete for protection.

To stop a train, there is hardly a better bumper than a pile of earth at the end of the track. But it is unsightly and perhaps not quite as positive as the Ellis bumper, which has been extensively used on Long Island. As a new departure, several concrete bumpers are soon to be built if the full-sized test proves satisfactory. It is fully anticipated that the car will be tested to destruction (Fig. 4).

For low platforms, old 8 x 16-in. trestle timbers or new 6 x 12-in. have been used for curbing, but with the replacement of the timber trestles by concrete arch or slab bridges or by pipes, the supply of the second-hand timber is insufficient, and hollow concrete curbing is now being used instead. The sections are 6 x 12-in. with a 3 x 8-in. core to reduce their weight. They are 10 ft. long with a $\frac{1}{2}$-in. rod 9 ft. 6 in. long in each corner. The curbs and posts are molded in the stock yard and taken thence to the site, set to grade and line and bolted together at a cost of about 50 cents per lineal foot of curb.

FIG. 5.—QUENTIN STREET BRIDGE; KEW GARDENS STATION IN CENTER BACKGROUND. THE FENCE IS MADE OF POSTS 7 IN. SQUARE AND SLABS 3 IN. THICK, 10 FT. LONG, 4 FT. 6 IN. HIGH, WITH MIDDLE SUPPORT.

One of the most satisfactory of the small uses developed for concrete is for fences; right-of-way posts, for instance, can be made which cost only about 45 cents each; the cost of the fence per lineal foot, spacing the posts 20 ft. apart and using woven wire, is as little as 10 cents (Fig. 5). An elaboration for bridges consists in the use of galvanized iron pipe instead of the wire, but under certain conditions neither of these types are sufficiently exclusive. The small boy is at times strongly tempted to drop or throw stones at trains passing beneath a bridge, but a high concrete fence at the sides of the bridge shuts off his view of the results, and to that extent discourages his efforts; the

passing of trains under a bridge is also more or less disturbing to horses; the enclosure of freight yards by a visual barrier is important. For these reasons, the Long Island Railroad has built a great deal of solid slab concrete fence, spacing the posts, which are about 7 in. square, approximately 10 ft. apart and erecting a 3-in. slab between posts. Former practice has been to first cast the posts in place, then the slabs in place, but it is difficult to hold reinforcement in the middle of such a thin slab and at the same time get the concrete thoroughly into the forms. In a few spots

FIG. 6.—BROADWAY, FLUSHING, N. Y. LARGE POSTS CAST IN PLACE; PRECAST PANELS THEN SET UP; SMALL POSTS CAST LAST. STATION PLATFORM RESTS ON CONCRETE PILES.

the steel is too near the surface and has rusted enough to flake off the thin concrete covering. A contract involving 12,000 feet of fence is therefore being executed now, premolding the slabs flat on the ground, the slab rods being merely laid into the half-filled forms, which are then filled to the top. The method of construction is to caste alternate posts in place, then set up between each pair of posts two premolded panels with their ends in expansion slots cast in the posts; and finally to pour the remaining alternate posts encasing the adjacent ends of the slabs.

Concrete has proved very serviceable in the construction of electric sub-stations on the Long Island Railroad. The founda-

tion walls are provided with all sorts of recesses and openings for the passage of conduits, and many of the conduits are built directly into the walls and floors; the intricacy of the conduit systems, however, is all covered by the concrete envelope and the iron pipes serve to some extent as reinforcement, although they are not assumed to have any value in this respect. For the structures to carry the switches, brick has heretofore been used, but the expense of setting pressed brick with the desired accuracy was so much greater than ordinary brick work that concrete slabs cast in the same manner as the fence slab have been used with great success for this detail.

Concrete pipe for culverts has not proven altogether desirable. It is so heavy in the large sizes that a crane is required to handle it and the consensus of opinion appears to be that cast iron is cheaper in place than concrete pipe.

For cattle guards it is found that triangular sections 5 in. high and 8 ft. long, reinforced with three $\frac{1}{4}$-in. rods, are very satisfactory. Niches are left in both edges about 6 in. apart so that one spike suffices to hold both adjacent guards. The ends are rounded off to prevent unsightly breakage. These are giving better service than either the old wooden guards or the newer vitrified sections.

The waterproofing of concrete has probably given as much trouble and requires as careful consideration as any question having to do with this material. Several years ago we considered an asphalt pavement sufficient protection for highway bridges, but time has indicated that this is not so. If the structures are to be made as nearly permanent as possible the concrete should be very carefully mixed, with a maximum density as the object, and deposited with a minimum number of joints, either horizontal or vertical; but in addition to this, thorough waterproofing is considered necessary, preferably by the so-called membrane method, generally using five-ply felt and pitch; two layers of felt, then an intermediate layer of reinforced felt, in order to prevent the cracking of the membrane through expansion or shocks, then two more layers of felt. This method is now used for both highway and railroad bridges; in the case of the former, the asphalt paving blocks which are generally used are laid directly upon the waterproofing in a half-inch bed of stiff mortar, and the joints either grouted or poured with pitch, while with the

railroad bridge decks the waterproofing is protected from being punctured either by the tools of laborers or the pounding of ballast under traffic, by using a layer of ordinary brick, likewise laid in a cement mortar bed. In the case of deck bridges, the fabric and brick are carried up the inside of the parapet to at least a foot height above the deck, or to the top of rail, whereas with the half-through construction the waterproofing is carried from the top of the slab up an inclined surface to flash under either the top flange of shallow girders or under an obtuse angle or bent plate riveted to the webs of deep girders at about the elevation of the top of rail.

The method of handling concrete which has for some time past been favored by the Long Island Railroad Construction Department is to set up the mixer in a pit, so that the ingredients can be either wheeled to it in barrows or delivered in small narrow gage cars which are loaded with stone, sand and cement directly from the cars in which they are delivered. After mixing, the concrete is hoisted in a bucket and discharged into a hopper, being fed thence into buggies and wheeled along the various runaways to the desired point. Spouting has been tried, but with a not very wet mix has not proved as satisfactory as the method in vogue.

By far the largest concrete job ever undertaken by the Long Island Railroad is the four-track tunnel 3,530 ft. long at East New York. Two tubes will be for Long Island Railroad trains to Manhattan Beach, and two will be for the New York Connecting Railroad. This work involves 100,000 cu. yd. of concrete, reinforced with 2,400 tons of steel rods. It will be probably all executed in open trench.

The appearance of frame structures is generally inferior to that of concrete structures for two reasons: First, if the maintenance of the former is curtailed the appearance is greatly marred; and second, ordinarily the lines and surface finish cannot be made as pleasing as with the concrete. The usual practice on the Long Island Railroad is to rub the surface with a carborundum brick, wetting with plain water during the operation. A brand of cement has been in use for two or three years which has given exceptionally white surfaces under this treatment and altogether it has proven decidedly satisfactory. For a bridge recently begun, foundations will have a surface finish approximating that used

for bridges built in the City of Philadelphia during several years past, but which will have architectural treatment in addition to the surface finish.

A highway bridge built two years ago near Cold Spring, well indicates the difference between the appearance of a frame and concrete structure. This bridge is carried by small abutments and intermediate piers to span a deep cut. The floor is of I-beams encased in concrete, only the exterior line of beams which carry the fence parapet having the arched soffit. Almost any steel bridge which could be built in the same situation would suffer by comparison.

Concrete lends itself very nicely to obtaining effects under

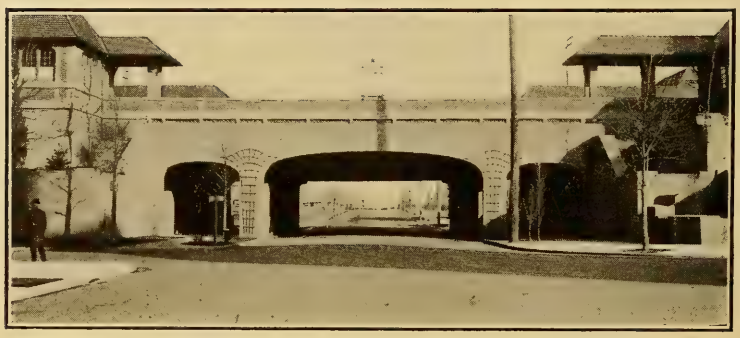


FIG. 7.—CONTINENTAL AVENUE BRIDGE. DECK PLATE-GIRDERS MASKED BY ARCHED CONCRETE FACING.

false pretenses; for instance, to save materials, piers may be pierced with arched openings with a very pleasing effect, although, of course, we know that no arch action takes place. Likewise a plate girder bridge which would not have a particularly pleasing outline may be hidden by a screen of concrete (Fig. 7) enhancing its appearance very materially at comparatively slight cost.

The Continental Avenue bridge is at Forest Hills, where the Sage Foundation Homes Company is developing a beautiful suburb. The combination of red-tiled roofs and brick panels, with the gray concrete of which the houses are built, makes a very pleasing effect, the warm tone being repeated in the stucco work on the bridge, by mixing in a considerable proportion of the red roofing tile broken into small bits.

RECENT APPLICATIONS OF CONCRETE AND REINFORCED CONCRETE IN THE CITY OF PITTSBURGH.

By N. S. Sprague.*

This paper relates entirely to the application of reinforced concrete to bridge repairs and new construction and is not so general in its scope as the title would seem to imply. It is not the purpose of the author to describe in detail the methods of design and construction of the large concrete bridges which have recently been built by the city or which are now in progress, but rather to call attention in a general way to some of the important features and problems which confront the engineer in the selection of the type of bridge best adapted to the site, traffic and other conditions, and the funds available for the purpose.

In the last four years the people of this city, and in fact the entire country, have shown a disposition to take a more active interest in public matters and from this interest there has sprung various civic organizations for the purpose of studying questions relating to the government and activities of the city. This general awakening of civic interest has stimulated action looking toward greater efficiency and economy in all branches of the public service and, therefore, when the citizens of Pittsburgh, by a referendum vote in 1908 and 1910, authorized the building of new and the rebuilding of some of the old bridges in the city, the engineers were confronted with the old and usual questions of utility, economy of maintenance and design, which were also being given special consideration for the first time in the history of the city by the people themselves.

Since concrete reinforced with steel is now recognized by most engineers as a suitable material of construction for bridges it becomes the duty of the bridge engineer to weigh the relative merits of this material with those of steel and to decide, after taking into consideration the other questions involved, which is

*Superintendent, Bureau of Construction, Pittsburgh, Pa.

best suited for the work at hand. Without attempting a detailed comparison of the relative merits of these two materials it must be borne in mind that while Pittsburgh is known all over the world as the Steel City and the center of the steel industry, yet its atmosphere, containing the smoke and gases produced in the manufacture of steel, is peculiarly injurious and a most destructive agent of steel bridges.

The city maintains within its corporate limits seventy-eight bridges of various types, with spans varying from 20 to 800 ft., having an approximate value of $5,000,000.

An examination of the records of the appropriations made by council for bridge repairs and maintenance from 1900 to 1907 will show that they were totally inadequate for the proper maintenance of these structures, the average yearly appropriation amounting to only one per cent of the total valuation of the property. Legislative bodies are frequently reluctant to provide funds for bridge repairs and maintenance, unless such repairs are absolutely necessary for the safety of the structure or cannot be deferred; I believe this is true of all legislative bodies to a greater or less extent, since the demands upon the public treasury are always in excess of the money available for such purposes.

The public official who is charged with the building of new and the maintenance of old bridges, and who must secure each year appropriations for bridge maintenance, is constantly studying those types of construction which, if incorporated into his designs, will reduce the cost of maintenance to a minimum.

It was, therefore, a question of maintenance and protection of the steel from the destructive agencies, which influenced the adoption of a new policy in 1907. The use of paint for the protection of steel bridges is expensive and unsatisfactory, requiring frequent applications to preserve the metal, and wherever possible those parts of the steel bridges most exposed have been encased in concrete and the new bridges built entirely of this material where local and economic conditions were favorable.

Steel Protection and Minor Concrete Bridges.

The first use of concrete by the city for the protection of steel was in 1908 and consisted in encasing in concrete some of the columns and struts of the Millvale Avenue and Twenty-eighth Street

bridges (Fig. 1), over the main line tracks of the Pennsylvania Railroad, to protect them from the injurious effect of locomotive gases. Where parts of the bridges were encased in concrete the proportions used were 1:2:4; the stone did not exceed $\frac{3}{4}$ in. in its largest diameter, and the concrete was reinforced where necessary with expanded metal and rods. This method of protection of the steel has since been used extensively in many other city bridges. The most extensive work of this character was the laying of cement sidewalks on the South Tenth Street, South Twenty-second Street, Negley Avenue, Aiken Avenue and several other smaller bridges, replacing the old sidewalks, which furnished practically no protection against the corrosion of the metal.

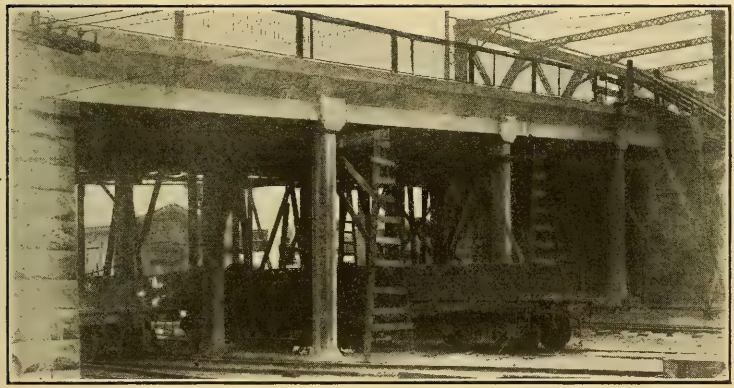


FIG. 1.—TWENTY-EIGHTH STREET BRIDGE, LOOKING EAST.

In 1909 several of the small wooden bridges over Saw Mill Run, in the West Liberty section of the city, had become unsafe and plans were made by the bureau for new bridges of reinforced concrete; this was the initial step by the city in building concrete bridges. These structures (three in number) were through girders of single span varying from 15 ft. 6 in. to 35 ft., and with roadways from 12 to 20 ft. in width, supported upon masonry abutments. The reinforcement of the girders consisted of structural shapes and was self-supporting, although falsework was required to support the concrete. The roadways were designed in the usual manner of beam and slab construction. The Graham and Blaine

Street foot bridges (Fig. 2), over the Pennsylvania and the Baltimore & Ohio Railroad tracks, were also constructed of reinforced concrete.

Description of Concrete Bridges Recently Constructed and Proposed.

The recent bond issues for public improvements included items for several large highway bridges, and of this number it was decided to construct the following of reinforced concrete:

Meadow Street over Negley Run.
Larimer Avenue over Washington Boulevard.
Atherton Avenue over the Pittsburgh Junction Railroad.
Atherton Avenue over the Pennsylvania Railroad.

Fig. 2.—Pine Street Bridge, Looking East.

Murray Avenue over William Pitt Boulevard.
Haights Run Highland Park entrance.

On account of the magnitude and prominence of these bridges special effort and study was made to secure graceful and artistic structures.

Meadow Street bridge (Fig. 3), was the first of these structures undertaken. It is 454 ft. in length and 50 ft. in width, with two 10-ft. sidewalks. The main span is 209 ft., with a rise of 46.14 ft., and consists of three arch ribs; the outside is uniformly 3 ft. 9 in. and the center 5 ft. in width, and all have a depth varying from 5 ft. at the crown to 6 ft. 2 in. at the springing line. These ribs are tied together with horizontal struts to provide additional lateral strength against wind pressure and possible indeterminate sec-

ondary stresses. The main piers are of cellular construction, the abutments U-shaped and the sidewalks are supported on cantilever beams. The bridge was reinforced throughout with deformed bars of medium open-hearth steel recommended by the American Railway Engineering and Maintenance of Way Association. The main span is a three-centered arch approaching very closely a parabola with the line of pressure practically coincident with the axis. Expansion joints were provided for the deck of the bridge at the abutments and main piers. The cost was $72,000.

The Larimer Avenue bridge (Fig. 4), with the exception of some minor details, finished last July, is the largest of its type that has yet been built. In appearance it resembles the Meadow Street structure, except it has two arch ribs instead of three. It

FIG. 3.—MEADOW STREET BRIDGE, LOOKING WEST.

is 670 ft. in length, 50 ft. in width, with two sidewalks each 10 ft. wide, and is 110 ft. above the ground. The main span is 300 ft. 4⅞ in. The arch ribs have a uniform width of 8 ft. and in depth vary from 6 ft. 6 in. at the crown to 11 ft. at the springing line and are 30 ft. on centers. The concrete was reinforced throughout with deformed bars, except in the arch ribs, where structural shapes were used. Crushed limestone was used in the concrete for all parts of the bridge above and gravel for all below the ground level.

To provide for expansion and contraction of the roadway, joints were located at the abutments, main piers, and over the third spandrel column from the main piers. This bridge contains 9,471 cu. yd. of concrete and will cost $170,000.

To relieve the congestion of automobile traffic on Center Avenue, between Craig Street and Liberty Avenue, it was necessary to widen and improve Atherton Avenue, which is parallel with and one block west of Center Avenue. In connection with this street improvement, two new bridges were required, one over the Pittsburgh Junction Railroad and the other over the Pennsylvania Railroad. The valleys spanned by the two Atherton Avenue bridges have considerably less depth than at Meadow Street and Larimer Avenue, and, therefore, the rise of the arches was correspondingly decreased, which was an important factor influencing the designs adopted. The bridge over the Pittsburgh Junction Railroad has a total length of 418 ft., is 60 ft. in width and has two 12-ft. sidewalks. The main span has two arch ribs

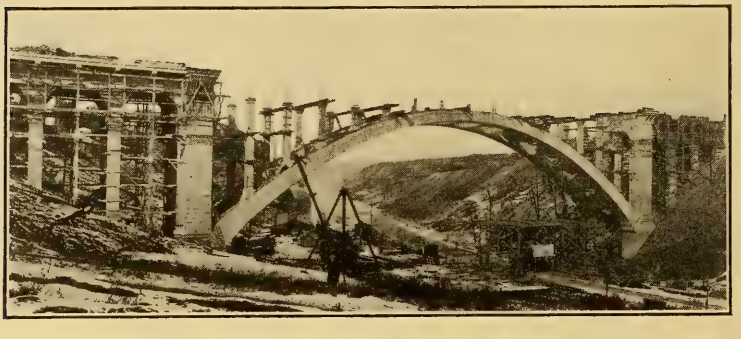


FIG. 4.—LARIMER AVENUE BRIDGE, LOOKING WEST.

14 ft. in width, 3 ft. 6 in. in depth at the crown and 7 ft. at the springing line, 37 ft. 6 in. on centers with a clear span of 168 ft. 6 in. and a rise of 32.5 ft. At the west end of the bridge there is one span of 60 ft. and at the east end a span of 45 ft.; both arches have twin ribs the same distance on centers and of the same width as the main arch. The main piers are solid instead of the cellular form of construction used in the other bridges. This was necessary to withstand the thrust of the 60 and 45-ft. arches which spring from an elevation 22.8 ft. above the springing line of the main arch. The spacing of the main arch ribs in this structure is such that the outer faces are flush with the facia girders under the coping, thus obviating the necessity for supporting the sidewalks on cantilever brackets such as was used in the Meadow

Street and Larimer Avenue bridges. The concrete in this bridge was reinforced with deformed bars, except the main arch ribs, in which structural steel was used. This bridge contains 7,960 cu. yd. of concrete and 626,800 lb. of steel reinforcement; it is now practically finished and will cost about $93,000.

The other bridge on Atherton Avenue, over the Pennsylvania Railroad (Fig. 5), is unlike in appearance and design any of the other bridges before mentioned. This valley is wide and comparatively shallow; furthermore, the center line of the bridge and of the railroad make an angle of 62° 45′, which necessitated skew arches. The bridge is of reinforced concrete, is 380 ft. in length,

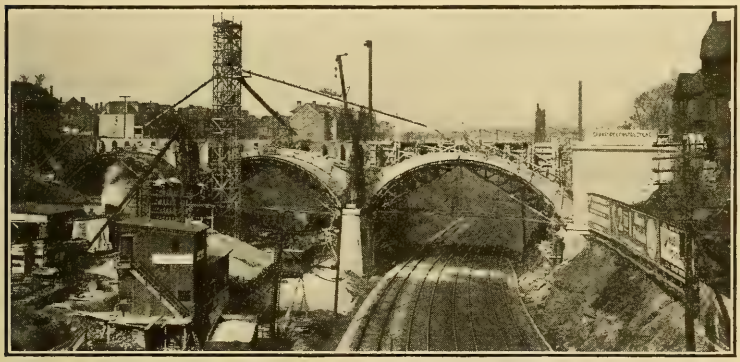


FIG. 5.—ATHERTON AVENUE BRIDGE OVER THE PENNSYLVANIA RAILROAD, LOOKING NORTHWEST.

60 ft. in width, with two 12-ft. sidewalks. It has three spans of 90 ft. each, with a rise of 21 ft. The arch rings are 2 ft. 6 in. in depth at the crown and 9 ft. at the springing line. The filling for the roadway will be granulated slag and is retained by spandrel walls. The concrete is reinforced with deformed bars, which are held in place in the arch rings by structural steel cross struts, to which the bars are attached. The arches were built in six longitudinal sections extending the full length of the bridge. One-half of the arch rings was first constructed; steel centering was used, so designed that upon the completion of one-half of the arch rings for the three spans the falsework was lowered on to rollers and then shifted into position for building the other half of the arch rings.

On account of the restricted space at each end of the bridge it was necessary to use a gravity system for placing the concrete, instead of cableways, as used on the other bridges. This bridge is practically finished, except the railing and pavement. It contains 8,700 cu. yd. of concrete and 519,000 lb. of steel reinforcement, and will cost about $95,000.

The Murray Avenue bridge is located in the Squirrel Hill section of the city, on the line of Murray Avenue, and crosses over the William Pitt Boulevard, replacing a steel viaduct now used exclusively by the Pittsburgh Railways Company. Since the appropriation for this bridge was obtained a Municipal Art Commission has been created who must approve all designs of city bridges costing $20,000 or more. This Commission desired a monumental and artistic structure. The plans for such a bridge were drawn and approved by the Art Commission, but could not be built within the appropriation and no additional funds could be secured. This first design was for two spans of 117 ft. each, with ring arches, spandrel walls and a filled roadway. This type of bridge, which embodies all the artistic features, is most extravagant in design and is open to the further objection of the difficulty in maintaining the roadway, due to the settlement and drainage of the spandrel filling. To build this bridge would require 9,400 cu. yd. of concrete and 834,000 lb. of steel reinforcement.

The detail plans for another design are in preparation. This revised plan calls for a reinforced concrete arch bridge 375 ft. in length, 60 ft. in width, with a paved roadway and two 10-ft. sidewalks. The main span is 210 ft., with a rise of 35 ft., and consists of two arch ribs each 10 ft. in width and 33 ft. on centers. The depth of the ribs at the crown of the arch is 5 ft. 6 in. and 11 ft. at the springing line. The floor system is similar in design and construction to those previously described and the same general scheme of steel reinforcement is to be used. This structure eliminates the question of settlement of the spandrel filling, simplifies the drainage of the roadway, contains only 4,500 cu. yd. of concrete and 400,000 lb. of steel reinforcement, and will cost about $40,000 less than the first design.

The first step looking toward the building of a suitable entrance to Highland Park from Butler Street was taken when an appropriation of $150,000 was made for rebuilding the Haights

Run bridge. This bridge will be located on the line of Butler Street, over Haights Run, at the intersection of the boulevard, park drive and the Nineteenth Street bridge leading to Sharpsburg over the Allegheny River. This bridge, therefore, must be first utilitarian, and second, it must harmonize in appearance with its surroundings. For this location it is proposed to build a reinforced concrete bridge with a single span of 195 ft. and a rise of 25 ft.; the width will be 60 ft., with two 12-ft. sidewalks and a 36-ft. roadway, supported on two arch ribs each 12 ft. in width and 32 ft. on centers. At the west end of the bridge it is proposed to build of concrete and stone two large ornamental pylons, and special attention will be given to the design of an appropriate railing. This bridge will be built in accordance with a comprehensive plan for improving the Park entrance which has been developed by the Bureau of Construction. Detail plans are now being drawn and it is expected that construction work will be started early in the spring.

Economy and Problems of Design.

The design of all these bridges has been greatly limited by the amount of funds available and it was therefore necessary to exercise the strictest economy and sacrifice some of the artistic features. The Larimer Avenue bridge, represents the highest type of economic development consistent with the climatic conditions which obtain in this section of the country. The two-ribbed arch for long spans has much in its favor with respect to economy of materials and simplification of design. It is true that the floor system for the twin arches must be heavier and the excess of materials used in its construction must be credited to the material saved by the omission of the middle rib and columns; on the other hand, there is a very decided reduction in the amount of forms required, which is perhaps the most expensive item of construction. If the arch is made of three ribs, the middle rib carries the greatest load and must be made stronger than the outside ribs, and this condition of unequal loading produces indeterminate temperature stresses, due to the variation in the size of the ribs, which tend to produce cracks in the floor system.

The topography of Pittsburgh is favorable to the construction of large concrete bridges because of ravines and valleys, where

the differences in elevation permit arches of an economic rise, with a good rock foundation relatively near the surface of the ground. Nature has thus endowed the city in this respect with peculiar advantages not enjoyed by most large cities.

All arches were designed disregarding the use of the rib reinforcement, which to some engineers may appear extravagant; but since the number of long-span concrete arches is small and of comparatively recent construction, the opportunity to observe their behavior under various conditions for a reasonable period has not been possible. Under these conditions the ribs were designed only for compressive stresses and the introduction of the steel in the ribs was simply to provide an additional factor of safety, more rigidity in the whole structure and greater elasticity in the ribs.

One of the greatest practical difficulties in reinforced concrete construction of this nature is to secure the proper spacing of the steel rods and to prevent their displacement in the process of concreting. Large steel rods such as are required in massive structures are most difficult to bend and handle, and these difficulties have been overcome by the use of structural shapes in the arch ribs, which insure their correct location in the concrete and facilitate erection.

Economy of design in conjunction with proper artistic effects, in the construction of concrete bridges, is most difficult to secure, and often prevents the attainment of the best architectural results. We are compelled to keep within the appropriation and must sacrifice and limit artistic and ornamental features.

In making plans for the large concrete bridges which have been described, the principal dimensions and general type were largely determined by the profile of the ground, and required certain forms of construction which could not be changed, however desirable a change might be in order to give the structure better lines. The architecture of some of these bridges, broadly speaking, consisted of two parts; first, the best treatment possible of such portions as were fixed by design or local conditions and could not be changed; and second, embellishments which added nothing to the strength, but greatly to the grace and beauty. The first part adds little additional cost, but the second will in general greatly increase the cost. Having satisfied the utilitarian require-

ments, many modifications are frequently required to produce an artistic structure. Theoretically the best manner of procedure is the adjustment of the type of bridge to the site under consideration. Graceful and pleasing lines can be adopted in all cases, even where the necessity for the strictest economy prevails. Where funds are available and where conditions make it desirable, strict conformance to architectural features and precepts are most desirable. If only the economic features of design were considered, ignoring the æsthetic entirely, this practice would result in one type of bridge, thereby subjecting the designer to criticism for similarity of design and for disregarding the excellent opportunities to give character, dignity and individuality to each structure.

Workmanship and Construction.

However carefully and accurately a concrete bridge may be designed, this work may be set at naught through its construction by incompetent, inexperienced and unskilled contractors. A large bridge is frequently an imposing and monumental structure, viewed daily by thousands of people; this fact emphasizes the importance and necessity of the engineer requiring the same degree of excellence of workmanship and skilled management in the erection of a bridge as the architect demands in the execution of his plans for some monumental type of building; the problem is more difficult of attainment for the engineer than for the architect, since in concrete bridge construction most of the work is performed by unskilled labor, with materials which are more difficult to handle. A large concrete bridge is composed of a number of large monolithic sections, which must of necessity be built at different times, under varying conditions, and all of which demand the utmost attention and watchfulness to secure adequate bracing of the forms to maintain their true and correct alignment. The bulging of a form or the unnoticed escape of grout from the concrete are defects which may not impair the strength of ordinary construction and which are easily remedied in most forms of concrete work, but in bridge work these accidents may ruin the appearance of the bridge.

Closely related to the importance of good workmanship is the question of the surface finish of the concrete. This may seem

to some engineers like a relatively unimportant matter and a detail which is unworthy of serious consideration, and this is undoubtedly true of some structures; but the writer is convinced that if the general use of concrete for bridges, buildings, towers, etc., is to continue, it will depend in no small measure upon the ability of the engineer to produce effects in surface finish which will compare favorably with other building materials. There is a large and profitable field for investigation and experiment along this line. Because concrete has a smooth surface of uniform color, and is lacking in character and individuality, it is frequently desirable to roughen this surface to expose the aggregate or to incorporate in its mixture aggregates other than those ordinarily used. The writer is of the opinion that this will be the ultimate solution of this most important problem, rather than the application of external treatments. In some cases the writer has been successful in obtaining good results by bush-hammering the concrete with a hammer having about eleven points to the square inch. This method of finishing the surface of the concrete requires the use of broken stone aggregate instead of gravel and considerable skill on the part of the workmen to obtain a uniform finish and to avoid chipping the corners.

With a view to improving and producing a greater contrast and a more pleasing appearance in concrete surfaces, some experiments were made with blocks of concrete, using broken red bricks, brick dust and Ligonier limestone screenings for the aggregate instead of gravel and broken stone. Three blocks of concrete were made using different aggregates.

Block No. 1 was composed of 1 part of Portland cement, 2 parts brick screenings varying from dust to $\frac{1}{4}$ in. and 4 parts of broken brick graded from $\frac{1}{4}$ in. to $1\frac{1}{2}$ in.

Block No. 2 was composed of 1 part of Portland cement, 2 parts limestone screenings graded from dust to $\frac{1}{4}$ in. and 4 parts of broken brick graded from $\frac{1}{4}$ in. to $1\frac{1}{2}$ in.

Block No. 3 was composed of 1 part of Portland cement, 2 parts Allegheny River sand and 4 parts broken brick varying in size from $\frac{1}{4}$ in. to $2\frac{1}{2}$ in.

Twenty-four hours after pouring these blocks the forms were removed and they were allowed to season for a period of 30 days. The surface was then bush-hammered with pneumatic hammers

having about eleven points to the square inch. The outside skin of the concrete was removed to expose the brick.

Block No. 1 presented a uniformly dark red brick color.

Block No. 2 presented a mottled surface of gray and red.

Block No. 3 presented the usual gray color of concrete, mottled with red, with little contrast, and was unsatisfactory.

The result of these experiments were so satisfactory that it was decided to incorporate concrete made in this manner into certain parts of the Hoeveler Street bridge (Fig. 6). This bridge, which is now practically finished except for paving, is located in the East Liberty section of the city. It is a small bridge 154 ft.

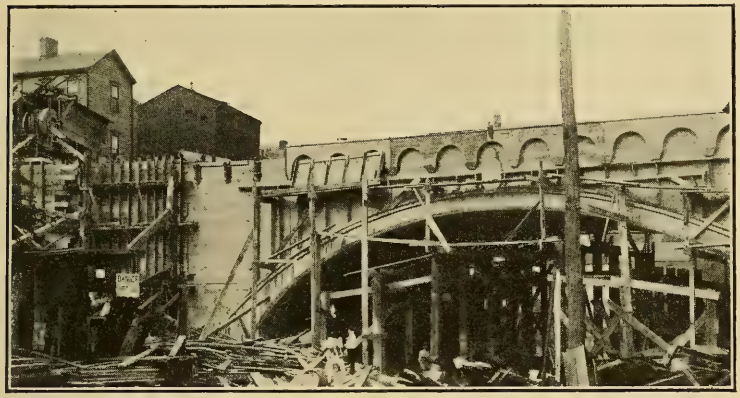


FIG. 6.—HOVELER STREET BRIDGE, LOOKING NORTH.

in length, having a single arch ring 60-ft. span with spandrel walls and a filled roadway. The specifications required the contractor to use concrete of various aggregates for the different parts of the bridge, as follows:

"The concrete for the arch abutments below the ground line and for the footings of the abutment side walls shall be composed of 1 part Portland cement, $2\frac{1}{2}$ parts sand and 5 parts gravel.

"The concrete for the arch ring above the ground line shall be composed of 1 part Portland cement, 2 parts sand and 4 parts broken stone.

"The concrete in the abutment side walls above footings, except posts at end of walls and the ornamental top and inside face of parapet walls, also the spandrel walls over arch, except ornamental top and inside face of parapet walls and seal over crown of arch shall be composed of 1 part Portland cement, 2 parts brick dust and 4 parts broken brick.

"The concrete for the posts at ends of abutment sidewalls and the ornamental top and inside face of parapet walls over abutment sidewalls and spandrel walls, excepting seat at crown of arch shall be composed of 1 part Portland cement, 2 parts Ligonier screenings and 4 parts broken brick.

"The concrete for the seals over the crown of the arch shall be composed of 1 part Portland cement and 2 parts Ligonier screenings."

The following specification was used for the broken brick, brick dust, broken stone and Ligonier screenings:

"*Broken Brick.*—The broken brick used as aggregate shall be crushed, hard burned, dark red brick of uniform color. The crushed brick shall be of graded sizes, the largest of which must pass through a one and one half inch ($1\frac{1}{2}$ in.) mesh screen and the smallest must be retained upon a one-fourth inch ($\frac{1}{4}$ in.) mesh screen.

"*Brick Dust.*—The brick dust shall be made from the same quality of brick as broken brick and shall pass through a one-quarter inch ($\frac{1}{4}$ in.) mesh screen.

"The use of soft or yellow brick for either broken brick or brick dust will not be permitted.

"*Broken Stone.*—The broken stone used as aggregate in concrete shall be fragments of hard, durable stone broken so as to pass through a screen having one inch (1 in.) mesh and be retained on a screen having one-quarter inch ($\frac{1}{4}$ in.) mesh, and be evenly graded from fine to coarse.

"*Ligonier Screenings.*—The Ligonier screenings used in the ornamental scrolls and wearing surface of sidewalks shall pass through a screen having two (2) meshes per lineal inch and at least seventy per cent (70%) be retained on a screen having ten (10) meshes per lineal inch."

Opinions seem to differ as to the result of this experiment. From an artistic point of view it has been pronounced a great success, although one eminent railroad engineer who viewed the work disapproved of the whole idea. The writer thinks that something has been achieved that will at least lay the foundation for future designs and which has overcome in some degree the objection raised by many to the general use of concrete for building purposes, that is, the difficulty in obtaining a suitable contrast in its texture.

RELATIVE COSTS.

With respect to the relative difference in first cost between a concrete and a steel bridge, most any conclusion can be reached, depending upon the assumptions made in making the comparison. For example, let us assume that a steel arch bridge 50 ft. in width,

with masonry abutments, could be built for $150,000 on the site of the Larimer Avenue bridge; it can also be assumed with equal propriety that a steel bridge of the cantilever type and of the same general dimensions would cost a somewhat smaller sum; and again, a steel bridge of the viaduct type could be built for much less than either of the others. The question then arises, on which type of steel bridge is the comparison to be based, before any comparison of cost of steel and concrete is possible; and the answer will greatly affect the result. The same reasoning holds true of reinforced concrete bridges, but perhaps is not subject to such variations. It would be manifestly unfair to compare the cost of a concrete bridge, like the original design for the Murray Avenue bridge, with its two spans, center pier, ring arches, spandrel walls and all the other features which make it artistically correct, with a steel bridge of the viaduct type. The comparison, therefore, should be made by selecting that type of steel bridge designed for the same loading, which compares favorably in appearance and in other respects with a concrete structure.

Let us assume that a concrete bridge can be built in a given location similar to the Larimer Avenue bridge for $170,000; that a correspondingly good steel structure can be built for $150,000; that its life with proper maintenance is fifty years, and make a comparison on this basis:

STEEL BRIDGE.

Annual charge for repainting, assuming the bridge is repainted once in five years at a cost of $5,000	$1,000.00
Annual charge for repairs 5 per cent of the first cost of bridge	750.00
Annual interest charge at 4 per cent on the investment	6,000.00
Annual sinking fund for renewal at end of fifty years at 4 per cent	982.50
Total annual charge for steel bridge	$8,732.50

Now if the concrete bridge cost $170,000, or $20,000 more than the steel structure, the only charge which can properly be made against it is interest on the cost of the investment; this, computed at 4 per cent, would amount to an annual charge of $6,800. Assuming this reasoning is correct, it appears that an expenditure of $218,300 would be warranted for the building of a concrete bridge, as against $150,000 for a steel structure.

In the above comparison it is assumed that both bridges would have the same repairs to the roadway and this item is therefore neglected. In addition to the financial advantage of the concrete bridge over one built of steel, there is the æsthetic difference for which no numerical value can be assigned, but which, nevertheless, is an important element in the comparison.

DURABILITY OF REINFORCED CONCRETE BRIDGES.

The use of concrete as a building material dates back to the time of the Romans, but its use in conjunction with steel is of

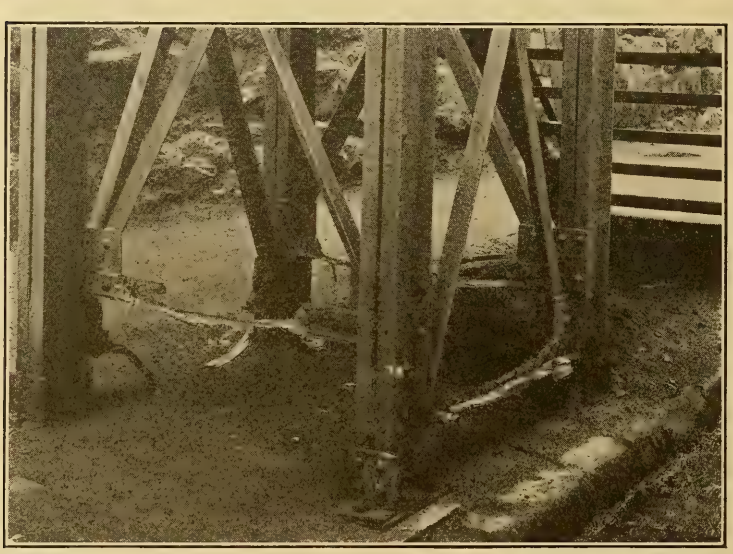


FIG. 7.—BLAINE STREET BRIDGE, FOOT OF STAIRWAY TOWER.

comparatively recent origin, and hence it has not been possible to determine the effect upon the concrete and steel reinforcement of these forces which would naturally tend to destroy and shorten the life of the structure. Since the advent of reinforced concrete, many instances are recorded where structures of this material have been rebuilt or altered, affording an opportunity to observe the condition of the steel reinforcement. In connection with recent repairs to the Smithfield Street bridge, some concrete used for protective purposes was removed from the end posts which had been

in place for twenty-two years. A careful examination of the steel showed no indications of corrosion and it was apparently in as good a state of preservation as when erected. From such observation, tests and experience, it is proved that concrete furnishes a good protection to steel against corrosion (Fig. 7); but the steel may be subject to the destructive influence of electrolysis. The danger of electrolysis in a bridge could easily be avoided by designing a properly connected steel reinforcement.

The writer has heard many engineers give expression to the thought that reinforced concrete structures will endure forever. This statement may be true of certain structures which are removed from and not subjected to the action of the elements or do not have to resist impact, but a concrete bridge is not so fortunate in this respect; it is necessarily exposed to the elements subjected to great variations of temperature incident to climate and must withstand the shocks and jars of large and heavy street cars. Should cracks develop in the bridge (and they are not uncommon, however carefully it is designed), and permit the entrance of moisture, the effect of continued freezing and thawing upon the concrete, year after year, and the possible corrosion of the steel reinforcement is difficult to foretell. In any event, these are factors that should be considered as intimately related to the life and durability of concrete bridges.

As an evidence of the durability and economy of concrete bridges, it is a significant fact that many of the large railroad systems of this country, and also a number of the state highways commissions, have adopted, wherever practicable, the use of concrete instead of steel for bridge construction. Such failures of concrete bridges as already have been recorded were due to improper foundations, faulty design or construction rather than to any inherent weakness in or disintegration of the material itself.

The writer is of the opinion that the use of concrete for bridge construction has passed the experimental stage; and furthermore, its use for such purpose has been satisfactorily demonstrated. The results are sufficient to justify engineers in the belief that the life of concrete bridges will at least equal and probably exceed that of steel structures, with a very substantial saving in the cost of maintenance.

DISCUSSION.

Mr. Cohen. MR. A. B. COHEN—I would like to ask the speaker what provision was made for expansion in the Larimer Avenue bridge and was the steel of the floor system run through the expansion joints?

Mr. Sprague. MR. W. S. SPRAGUE.—In the Larimer Avenue bridge there are expansion joints in the roadway at the third panel-point out from each of the main piers. The expansion joints for each of the approaches are located at the abutments and main piers. Expansion joints were provided at each panel point for the sidewalk slab and facia girder. There was a distinct joint left there. Double floor beams were provided at the third panel point. A sliding bearing composed of two bronze plates was also provided. The expansion joint extended the full width of the bridge.

Mr. Whited. MR. WILLIS WHITED.—There are one or two points to which I would like to call your attention regarding the design of the Larimer Avenue bridge. It is not quite correct to say that the bending stresses were ignored in designing the reinforcement. Mr. Sprague probably has reference to the shearing stresses, which are in no case heavy, and the concrete is amply strong to take care of them. There were, however, bars inserted at the bottoms of the columns where they connect with the main arch ribs, to resist the shear due to the column loading. The reinforcement is amply sufficient to resist any bending stresses that can be produced, although in no case were any tensile stresses found which exceeded the strength of the concrete. The main purpose of the reinforcement was to reduce the weight of the arch ribs.

Regarding the esthetic treatment of the structure, the intention throughout was to make the design simple, dignified, well adapted to the conditions and have an appearance of strength and stability which would produce a restful effect on the beholder. The aim was to use only enough ornament to bring out the various features of the design and avoid any sameness that would become wearisome. While a symmetrical structure is generally more

beautiful than an unsymmetrical one, the shape of the ground would not permit this, and a strained attempt at symmetry would rather tend to emphasize its lack. Mr. Whited.

As the railing is regarded by nervous people as the only bulwark between them and death, and by all as the only protection for children, it must be strong both in appearance and reality, and be so constructed that children cannot get through it and so that it is as difficult as possible for them to climb over it, special care being taken to prevent venturesome boys from walking on top of it. It is very important to make the effective width of the bridge as great as possible, without extending any part of the structure beyond the street line; for that reason the railing posts are narrower than on many bridges. A certain amount of ornamentation is suitable on the railing, and provision is to be made for some ornamental lamp posts, which will add much to the appearance of the bridge.

THE DEVELOPMENT OF CONCRETE GRAIN ELEVATOR CONSTRUCTION.

By R. P. Durham.*

It is the purpose of this paper to give a general review of the beginning of concrete elevator construction and its subsequent progress, paying particular attention to the building of bin walls with moving forms, which is the feature making concrete elevator construction so different from other forms of reinforced concrete work.

Reinforced concrete grain elevator construction in the United States had its beginning in the very closing years of the nineteenth century. Prior to the starting of this form of construction in America, certain concrete grain storage bins had been built in Europe which were, in their main essentials, practically the same as those built later in this country, except that the individual bins were generally smaller, both in plan and in height, than the average size of bins built on this side of the water. Neither was the total size of the complete building what we should call large. The mechanical appliances for handling the grain exhibited considerable difference from those used here; and, perhaps most important of all, the method of carrying out the construction differed very materially from the means which is now almost universally employed in the United States.

In the years 1899 and 1900 the construction of reinforced concrete bins was started practically simultaneously by three engineers working independently and at different points. One building was not for storage of grain but should be mentioned here as being essentially similar to a grain storage building and being one of the earliest examples of this form of construction. This building was the cement storage plant for the Illinois Steel Company of Chicago, designed by E. Lee Heidenreich and constructed under his supervision. It consisted of four bins 25 ft. in diameter inside, with bin walls 53 ft. 6 in. deep, 7 in. thick for approximately one-half the height and 5 in. thick the

* Vice-President, John S. Metcalf Company, Ltd., Montreal, Canada.

balance of the way. Before actual construction of this work was started, F. H. Peavey, the well-known grain merchant of Minneapolis, sent a representative to Europe to investigate the progress which had been made there in the construction of concrete grain bins. A thorough study of the situation was made and on the return of the investigator, Mr. Peavey, with his engineer, Mr. Haglin, built as an experiment a single concrete bin adjacent to one of the Peavey elevators at Minneapolis. As this was successful, construction of a large reinforced concrete grain storage elevator in connection with one of the Peavey elevators at Duluth was proceeded with. Unfortunately a portion of this elevator collapsed the first time grain was put into it and this, in spite of care in loading and various means used for strengthening the structure, was repeated on more than one occasion thereafter. It is not necessary in the present paper to discuss the reasons for the failures further than to say that they were not caused by any inadaptability of reinforced concrete to this form of construction, but rather by unfamiliarity with the action of grain and the proper method of reinforcing to carry the pressures.

The third set of concrete bins built at this time was a small grain elevator for the George T. Evans Milling Company of Indianapolis, Ind., by the John S. Metcalf Company of Chicago. It consisted of four cylindrical bins 26 ft. in diameter inside and 47 ft. high. The interspace bin between the four cylinders was used for grain storage. These spaces were also used in the Peavey Elevator and it was, in fact, the use of these spaces, without full knowledge of how the grain in them would act, which primarily caused the various collapses.

So new was the construction of concrete grain bins that in taking the contract for the small elevator at Indianapolis, the John S. Metcalf Company were obliged to guarantee the building for eight years. The elevator was in every way satisfactory and may be called the first successful concrete grain elevator in this country.

The trouble with the Peavey bins necessarily retarded the development of concrete elevator construction. Grain companies and railways building terminal elevators, naturally enough, in the absence of full information as to the reasons for the failure, laid the trouble to the materials used.

The author believes that the next reinforced concrete elevator was one built by the Barnett and Record Company of Minneapolis for the Canadian Pacific Railway at Port Arthur, Ont. It consisted of nine cylindrical bins 30 ft. in diameter and 90 ft. high with bin walls 9 in. thick. The interspace bins were also used for storage and the elevator was in every way satisfactory. This elevator was completed in January, 1904.

In the meantime the construction of fireproof grain bins by using tile walls was progressing so rapidly under closely held patents that builders who were not in position to use this form of construction were continuing their investigations of the use of reinforced concrete as a competitor of tile. In 1904 the John S. Metcalf Company built for the Missouri Pacific Railway, at Kansas City, Mo., a reinforced concrete elevator of 720,000 bushels capacity. This consisted of one long line of ten cylindrical bins independent of each other except as to foundation, each bin being 40 ft. in diameter and 80 ft. high. From that time to the present by far the largest proportion of terminal elevators, and storage elevators for mills, have been built of reinforced concrete, gradually displacing construction of wood, steel and tile. The small country elevator is still largely constructed of wood, although the Macdonald Engineering Company of Chicago have recently built two or three country elevators of reinforced concrete and state they can build economically and rapidly of that material.

Owing to supposed difficulties in reinforcing rectangular concrete bins, there were for several years few concrete grain bins built which were not of cylindrical form.

About the year 1907 J. H. Tromanhauser designed and built a 500,000-bushel elevator at Goderich, Ont., in which the bins were rectangular in form. The walls in this elevator were designed so as to be thicker at the center of the span than at the points of support. In 1909 John S. Metcalf Company built for the Grand Trunk Railway, at Tiffin, Ont., an elevator with rectangular bins and straight walls 8 in. thick. Within the last two years a number of rectangular bin concrete grain elevators have been constructed; the most notable example being the recently completed 2,600,000-bushel elevator for the Harbour Commissioners of Montreal (Fig. 1).

There is at least one concrete elevator in the United States

with bins which are hexagonal in form. This is an elevator built some years ago by the Macdonald Engineering Company at Nashville, Tenn.

You now have had a general outline of the development of the concrete elevator business and it only remains to state in this connection that concrete, at least for terminal and milling elevator construction, has now almost entirely displaced not only wood, but steel and tile.

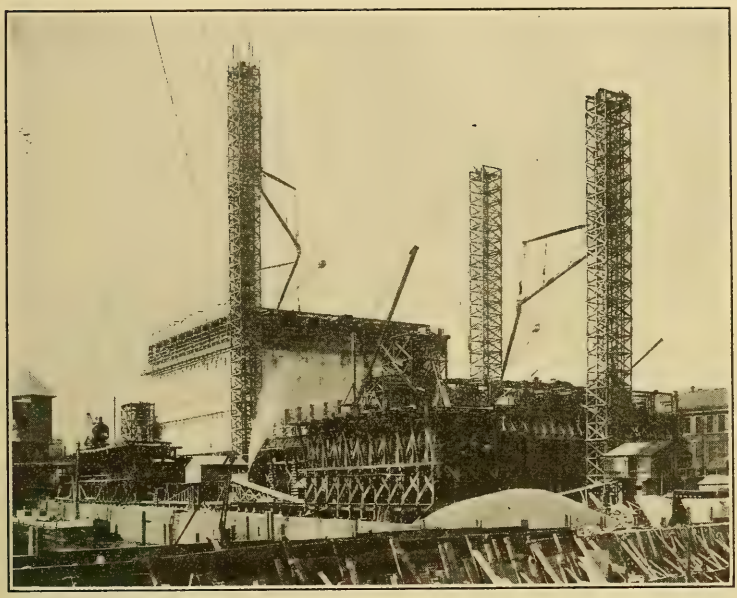


FIG. 1.—HARBOUR COMMISSIONERS' ELEVATOR NO. 2, MONTREAL, QUEBEC.

The displacement of wood is readily comprehended and is only a portion of the prevailing tendency toward fireproof and permanent construction. The ascendency of concrete over steel and tile arises from the fact that it can be more cheaply constructed and is, as regards steel, rightly considered by the underwriters a considerably better fire risk.

In the earlier days of concrete elevator construction, the superstructure above the bins, or what is technically known as the cupola, was built of structural steel with concrete floors and

roofs and generally a corrugated steel outside covering. Recently a number of elevators have been built with reinforced concrete cupola. It undoubtedly presents a better fire risk than the steel and a lower cost of maintenance. The question as to whether it can be built as economically and quickly as a structural steel cupola is one on which there is still room for argument. The cupola of a terminal grain elevator requires a considerably more complicated form of construction than the ordinary reinforced concrete warehouse building. The most prominent example of concrete cupola construction is at the new Montreal Harbour Commissioners' Elevator, and this cupola, whether in comparison with elevator construction or any other form of reinforced concrete building, is an instructive example of what can be done with concrete (Fig. 2). Not only are the columns, girders, beams, floors and roofs of concrete, but the curtain walls and even the elevator leg castings, 2 in. thick, are of this material.

There is nothing in connection with the construction of foundations and first story of a concrete grain elevator, and very little in the construction of the cupola, calling for special comment as to method of procedure. Aside from the fact that foundation loads are exceedingly heavy, and the item of the complicated construction sometimes necessary in the cupola, as already mentioned, the ordinary methods of concrete warehouse construction are in general followed in building such portions of grain elevators. The building of the bin walls has, however, developed a method of construction not used, as far as the author is aware, in connection with other buildings. This arises from the great height of bin wall of the same thickness, which early suggested that a short form which could be moved up continuously would be the most economical means to be used.

The earliest concrete bins, viz., those at South Chicago, Minneapolis, Duluth and Indianapolis, above referred to, were built by means of fixed or shutter forms. As far as can be learned, the first set of bins on which the moving bin form was used was the cluster of nine cylindrical bins built for the Canadian Pacific Railway at Port Arthur, Ont., in the latter part of 1903.

All moving bin forms have certain characteristics in common, the variations between those used by different contractors, or by the same contractor at different times and on different work,

being more in method than in principle. The form consists of horizontal framing pieces to which vertical sheeting is attached. The form may vary in height from 3 ft. to perhaps as much as 5 ft.,

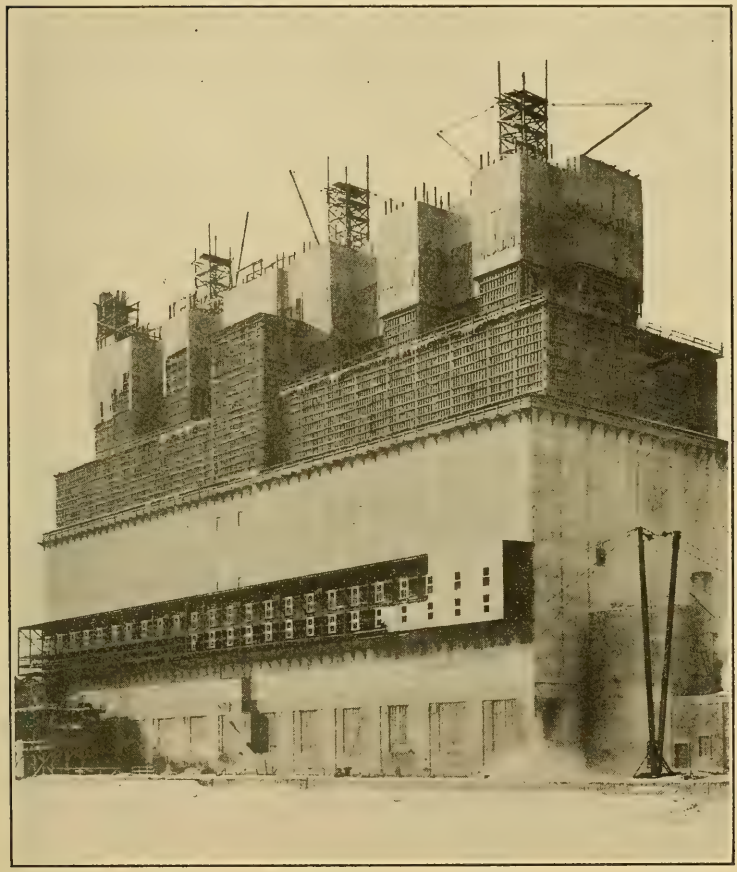


FIG. 2.—HARBOUR COMMISSIONERS' ELEVATOR NO. 2, MONTREAL, QUEBEC. Concrete cupola being constructed in mid-winter by totally enclosing and heating.

measuring by the length of the vertical sheeting, but is always a comparatively short form. It must, of course, extend along both sides of each wall, the forms on the two sides of the wall

being connected by vertical timber or steel yokes which are usually attached to the horizontal framing of the form. The sheeting is generally of wood dressed on the side in contact with the concrete. Galvanized sheet steel on a wood framework has also been used and in some cases the wood sheeting itself has been covered with steel, either on only the forms for the outside walls exposed to view or on all of the forms throughout the work. The purpose of covering the wood sheeting or lagging with sheet steel has been to make smoother walls, which would thus have a more presentable appearance and would also require less pointing up or patching. It has been found that the use of steel is not necessary if the raising of the form is carried on rapidly and continuously and a gang of pointers follows closely after the moving forms. In such cases the wooden sheeting has produced workmanship which is all that can be desired and in some ways is superior to that produced by stationary forms.

The principal difference in methods of building bin walls with moving forms is dependent upon the procedure followed for raising the forms. It is necessary, in order to obtain walls which are smooth and results which are economical, that the forms be raised continuously and that the movement be at a steady rate. The concrete at any particular point must have a reasonable length of time to harden before the bottom edge of the sheeting gets above it. This means that the progress in any one day is limited to a maximum of approximately twice the depth of the form, which insures that there will never be any exposed concrete which is less than twelve hours old. Assuming a 4 ft. form, this means that the movement must not exceed 4 ft. in twelve hours, and it is generally less than that; so that a very slow, but at the same time a very steady, upward movement is desirable. It will probably be asked, why not build forms with longer sheeting and jack faster. The answers are, first, that it is not desirable to put a much greater load on green concrete; and, second, that 6 to 8 ft. of concrete a day on a large building has been found to be the maximum which can be gotten into the forms, when the placing of reinforcing and other items tending to delay have been considered.

The Canadian Pacific Elevator at Port Arthur, built in 1903, was, as stated above, the first elevator, as far as the author is

aware, on which moving bin forms were used. The jacking was done by means of ordinary locomotive jack-screws. Brackets were fastened to the lower horizontal members of the forms and the jacks were set on vertical posts with the top of the jacks bearing against these brackets. As the forms were raised to the limit of each jack, another section of post was added below the jack and the entire form structure thus gradually raised from bottom to top by increasing the length of the post. The jack

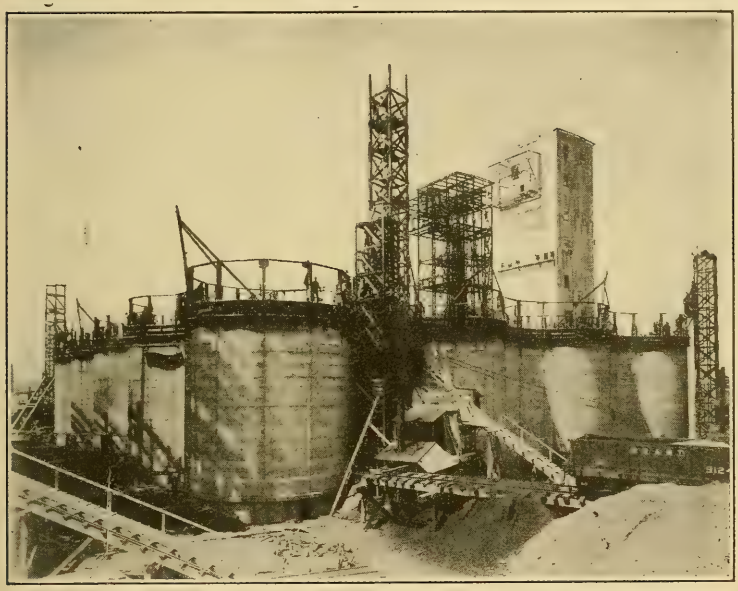


FIG. 3.—MOVING FORMS FOR CIRCULAR BINS. GRAND TRUNK ELEVATOR, TIFFIN, ONT.

posts were probably of varying length so that all jacks would not have to be released simultaneously.

Another method of raising moving forms, and that used by our company at the Missouri Pacific Elevator, Kansas City, and on one or two other contracts, was somewhat similar, except that the jack was placed at the bottom of the bin instead of at the top next to the form. Locomotive jack-screws were placed below the posts and the entire form structure and scaffolding gradually raised, the new sections of post being put in at the

bottom from time to time as the limit of travel of any jack was reached.

After this time, with the increased number of concrete elevators being constructed, and the increased number of engineers working on schemes for raising the forms, the method of jacking becomes more diversified. The style of jacks may, however, be divided into two general classes, the screw-jack and the pump-jack. The former depends on the travel of the nut on some form of threaded rod, either in the jack or in the bin wall; the latter depends on a toggle arrangement operated by a lever with pump-handle motion and working on a plain rod. Practically all builders now support the forms, not on a scaffolding between the bottom of the bin and the form, but on steel rods which are embedded in the bin wall. The jack travels upward on these rods, the jack itself generally being attached to the yoke, to the lower end of which the bin forms on either side of the wall are secured.

The question as to whether the pump-jack or the screw-jack is the better style is one on which experts disagree. Some who have used both kinds are very much in favor of the pump-jack; others who have had experience with both kinds stick to the screw-jack. We have experimented with the pump style but have never seen fit to adopt it in actual construction work. We have used two or three different styles of screw-jacks and have built about 10,000,000-bushels capacity of storage bins with the jack we are now using. In this case the vertical rod which is embedded in the wall, and on which the jack operates, is threaded the entire length with a double "V" thread about $3\frac{1}{2}$ pitch. The jack casting is attached to the yoke and consists of one supporting casting and a revolving casting with apertures for the jack levers. The revolving casting is set over a square nut on the threaded jack-rod. As the revolving casting is turned the nut climbs the jack-rod and the yoke and forms are raised. This jack enables us to make steady progress, is capable of exerting great power in case of the forms sticking, and is sufficiently fast. On two different sections of the Montreal Harbour Commissioners' Elevator, each section being about 150 ft. by 100 ft. in plan, we have run up 86 ft. of bin walls in less than 14 days, or better than 6 ft. per day. This is, of course, working night and day; for if a moving form is

stopped after it is once started a joint in the work is sure to show, and generally a small offset or shelf is left at the joint.

One company uses another type of screw-jack; in their case the vertical rod in the wall is a plain rod. The jack is, briefly, an extra heavy pipe about 2 ft. long and threaded on the outside. The jack-rod runs through this pipe, which is provided with a clutch at the lower end. The traveling nut operates on the pipe and when the limit of travel has been reached the clutch is released, the threaded pipe run to the other end of the travel and the operation repeated. They have secured good results with this jack. The Macdonald Engineering Company of Chicago, on the other hand, after trying various styles of jacks, have abandoned screw-jacks altogether and use only a pump-jakc. Another contractor has used both, of various styles, but of late has leaned to the pump-jack.

The rate of progress previously mentioned is possible only with a comparatively quick-setting cement such as is used in the United States and Canada. The standard British cement sets very slowly and the use of moving forms is unknown in Great Britain. Thus far, as the British engineers are slow to allow innovation, they have not been willing to permit the use of a quick-setting cement and a moving form.

From what has been said in regard to the speed at which bin walls can be built, it may be thought that there are no troubles in this business, but the author assures you that every builder of concrete elevators who has used moving bin forms has also had his griefs. Even when the forms themselves are practically perfect, there are other considerations which sometimes cause trouble. It is not possible to build a very thin wall with a moving form, the determining factor being the relation between the weight of concrete in the form and the friction of the concrete against the sides of the form. The area of form exposed to the concrete is the same no matter what the thickness of the wall, while the weight, of course, varies in direct proportion to the thickness. It is entirely practicable to build a 6-in. wall with moving forms, but that it is dangerous to go much below this unless very special precautions are taken to prevent lifting of the concrete. We have built concrete elevator legs 2 in. thick with moving forms and

gotten good work, but in that case we lifted the forms with chain blocks and went with comparative slowness.

The speed of the work and the freedom from trouble will also depend somewhat on the aggregate used and, it would seem in some cases, on the cement. Three elevator builders have had trouble with one particular brand of cement in moving forms and decline to use it further. A fourth has used it with success and is

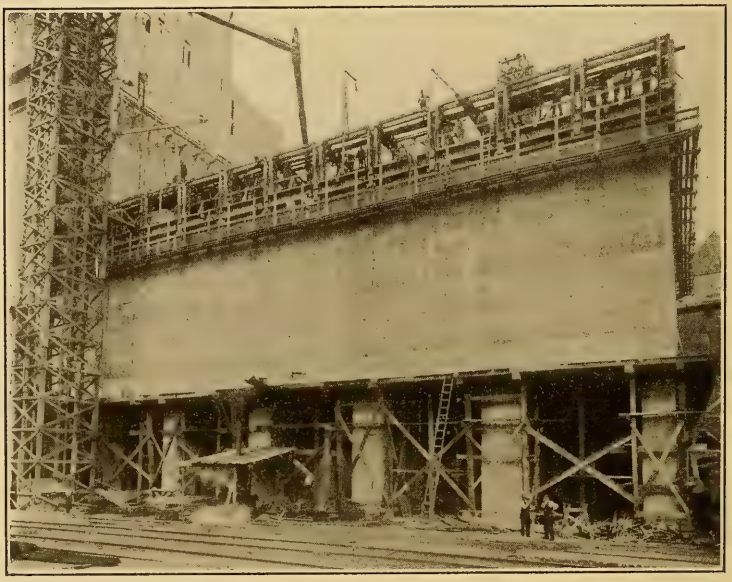


FIG. 4.—MOVING FORMS FOR STRAIGHT WALLS. HARBOUR COMMISSIONERS' ELEVATOR NO. 2, MONTREAL, QUEBEC.

not afraid of it. Whether the trouble arises from this particular cement having properties which cause it to stick to the forms more than other cements, or whether it sets before the forms have been moved and thus causes the top of the wall to break away from the portion below it, is a question no one, not even the cement manufacturers themselves, have been able to answer satisfactorily.

The question of aggregates seems to be a comparatively unimportant one. We have had good results with crushed-stone

concrete and with gravel concrete. The crushed-stone concrete has, perhaps, less tendency to lift in the forms, but on the other hand, on account of the likelihood of sharp corners of the stones catching in the wood of the sheeting when tamped, and thus being displaced as the form is raised, smoother walls are generally obtained with gravel concrete. We have successfully run bin walls with sand and cement alone, though in general we should consider this taking a chance because of the tendency of a cement mortar to stick to forms.

CONCRETE ELEVATORS.

By Barney I. Weller.*

Early History of Grain Elevators.

Elevators as a means of housing and handling grain did not make their appearance until the latter part of the last century. The first real elevator of which there is any record is the "cribbed" wood type and there are still a good many of these elevators in existence. This type is interesting when it is considered that at one time an elevator of nearly 4,000,000-bushels capacity was erected complete, and almost totally filled with grain in a period of forty-four days. Of course, lumber was plentiful, no expense was spared and no restrictions were put on the builder. As the price of lumber advanced it became necessary to look for other material; the elevator operator and owner seeking a material which would lower appreciably the very high insurance rate on wood.

The first fire-resisting elevators were built of steel, practically on the same plan as the old wooden structures which were rectangular in plan, had cribbed bins elevated on posts and were usually arranged to suit unloading conditions. Up to this point all storage and handling devices were carried under one roof, but it was then demonstrated that all machinery for unloading, handling and shipping could be more economically installed in a separate building called the working house. This was accomplished by having two or more parallel tracks alongside this house for unloading, which, of course, necessitated decreasing its length and thereby making it more economical; a separate building for storage could be erected having larger compartments than the working house. At about this time there came into common use in the construction of elevators, brick, tile and concrete, of which we will treat later.

* Purchasing Agent, James Stewart and Company, Chicago, Ill.

Classification of Grain Elevators.

Grain elevators in general may be classified under the following heads: Terminal, transfer, country or line houses, private or hospital. Of the above-named classes, the terminal is by far the largest. This type is to a greater or less extent under the supervision of the government, both as regards the classification of the different kinds of grain and also as to the weights of same.

Terminal houses are so called because they are usually situated on a lake or ocean port and receive their grain either direct from the farmer or through their line or country houses.

When grain is received by a terminal house it is given a grade by the inspectors, and either is shipped direct by boat or rail, or, as is usually the case, is stored awaiting future delivery. Part of this grain may have to go through different processes, such as bleaching, drying or cleaning. The operators of these terminal houses usually make a nominal charge per bushel for handling this grain, allowing a certain number of weeks, usually three or four, of free storage; if the grain is stored for a longer time, there is another charge per bushel for each month or fraction thereof.

The cleaning and drying, when necessary, is generally absorbed by the operators of the house.

Terminal houses are usually controlled by grain firms who also have their own line and country houses. These country houses vary in size from 10,000 to 50,000 bushels capacity, and are placed along railroads throughout the country where the grain is grown. These houses, after receiving the grain from the farmer, ship direct to their terminal house.

A transfer house is usually located on an ocean port and usually receives its grain by boat from a terminal house. The grain delivered to a transfer house is usually of a known grade, and is either delivered direct to the consumer or for export.

Private elevators are owned usually by milling concerns who control their own elevators and country houses of sufficient storage and handling capacity to satisfy the requirements of their mills.

Hospital elevators are usually small affairs and owned by private individuals who make a practice of buying inferior or damaged grain and putting it through the processes of cleaning;

bleaching, drying, scouring, clipping or other operations necessary to give it a higher grade or make it more salable.

METHOD OF HANDLING GRAIN.

The farmer either ships his grain direct to a terminal elevator, if he is not in position to deliver to a line house, or he may sell it to a commission merchant. If he delivers to a line house it is usually by wagon to a dump scale, under which is a hopper feeding an elevator leg, which elevates to the top of the house, where the grain is spouted into storage bins or direct to cars. The car is sealed and shipped, let us say, to a terminal house, where it is placed in the track shed directly over the receiving pit. From this the grain passes to a conveyor belt which delivers it to a receiving leg, emptying into a garner over a scale hopper, from which it is weighed; the weights being recorded by both the government inspectors and the operators.

This grain then is generally delivered to a belt which, by means of a tripper, delivers it either into storage or to boats or cars; in case it needs cleaning before it is placed in storage, it is spouted to cleaning machines, or if it needs drying, to the drier; after which it will then take the same course as the uncleaned grain.

Modern elevators are now built with a certain amount of storage in the working house, which is that part of the elevator which contains the elevating, conveying, cleaning equipment, etc. In addition to this there is generally required a certain amount of storage capacity which adjoins the working house. This additional storage is generally called the annex, and grain is delivered to this annex by means of conveyor belts running over tanks. These tanks are hoppered to make them self-cleaning, and the grain is removed by conveyor belts running through tunnels under the tanks to the elevator legs. Grain required to be shipped by boat is usually taken from the storage annex, weighed and spouted by dock spouts into the boat. If this boat delivers to a transfer house, which is usually the case, the grain is taken from the boat by means of a marine leg housed along the dock, which can be raised or lowered at will. This marine leg delivers the grain to a lofter leg, which in turn delivers to the scale and thence to storage or to boat or cars for reconsignment. In order to

facilitate getting the grain out of the hold, usually both ship and clean-up shovels are used; these work automatically, drawing the grain to the leg. In the doctoring of grain great progress has been made in the last decade. It is possible, by the use of sulphur in the bleacher, to whiten oats which are off color, thus giving them a much better grade; the beards also may be removed by a clipper, which enhances their value several cents per bushel, and in the up-to-date type of cleaning machines all foreign seeds of every description can be removed from the different classes of grain. The improved driers are also a great achievement. In all

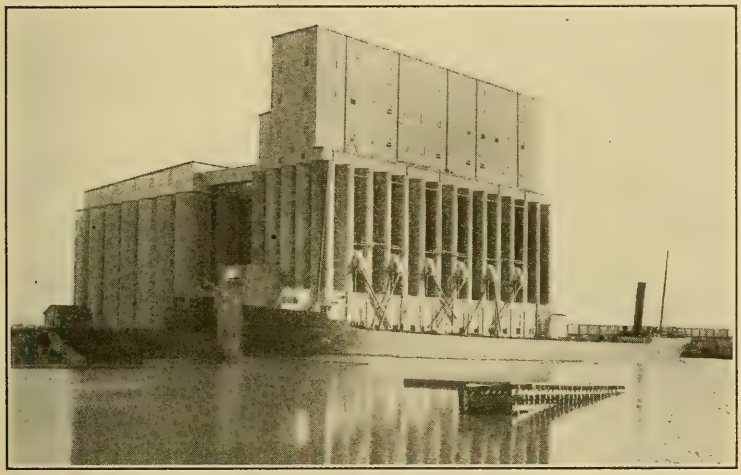


FIG. 1.—THE GRAND TRUNK PACIFIC TERMINAL ELEVATOR, FORT WILLIAM, ONTARIO. CAPACITY ABOUT THREE AND THREE-QUARTER MILLION BUSHELS.

modern houses dust collectors are also provided which do away with a great percentage of the dust and dirt always to be found in the elevator of the olden time.

It may be interesting to give an idea of how much work can be done by some of the elevators which lately have been constructed. The Canadian Stewart Company recently constructed an elevator for the Grand Trunk Pacific Railroad Company, at Fort William, Ontario (Fig. 1). In the track shed elevator there is room over the receiving pits for spouting over 20 cars at one time, and in a double shift of 20 hours it is possible to unload

over 600 cars of grain. A boat which can carry a cargo of 400,000 bushels of wheat can be loaded at this elevator by means of five dock spouts in about $3\frac{1}{2}$ hours. This house is equipped with 19 cleaning machines, each with a capacity of 3,000 bushels per hour, and a dryer of a capacity of about 2,000 bushels per hour; there are also 19 elevator legs, most of which have a capacity of 18,000 bushels per hour. There are ten 2,000-bushel hopper scales, each of which can be filled, weighed and unloaded in a little less than $3\frac{1}{2}$ minutes. The total capacity of this house is a little

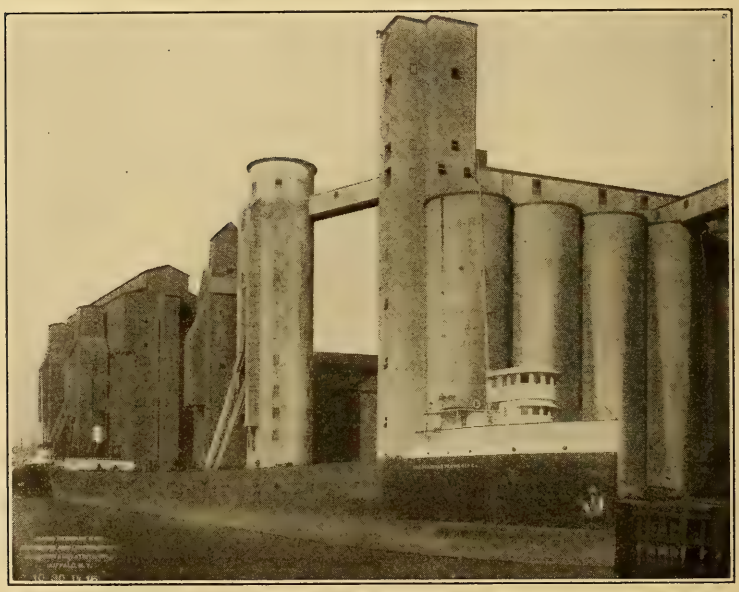


FIG. 2.—FRONTIER ELEVATOR AND MILL, BUFFALO, N. Y.

less than 3,750,000 bushels, of which the working house capacity is about 750,000 bushels.

In regard to power, it may be said that almost without exception all up-to-date houses are equipped with individual motors for the different machines, conveyor belts, etc., which has been proven satisfactory as to service and also economical.

Rubber belting of a high grade is used both for conveyors, and elevator legs; for transmission manila rope is used, gears and belting being obsolete; the former principally on account of the noise, and the latter on account of slippage.

James Stewart and Company recently erected a marine tower for the Washburn-Crosby Company of Buffalo, N. Y. (Fig. 2), cylindrical in form and 160 ft. in height, equipped with a marine leg and is by far the fastest on the great lakes, being able to unload grain at an average speed of 22,000 bushels per hour. The tower also contains automatic scales and delivers the grain direct to the company elevator by means of a conveyor belt running through a tunnel.

MATERIAL USED IN CONSTRUCTION.

The different materials of which elevators have been built are as follows: Wood, steel, brick, tile and concrete. These have been stated in the order in which they came into general use; at the present time few elevators are built of any material save concrete. Wood was found to be very expensive when insurance rates were taken into consideration. Steel is a high conductor of heat and there is on record an instance where four box cars lying in a track shed caught fire, resulting in the wrecking of a steel house of over 500,000 bushels capacity. Steel has been found impracticable because the shells of the storage tanks are very thin, offering little resistance to the fire in adjacent buildings, and much grain is damaged by the excessive heat. There have been a great many elevators constructed of brick, but it is usually too costly because of the thickness of walls necessary to suit reinforcing conditions, etc. Tile was used for a number of years, and even now elevators are sometimes built of this material; the main objection to its use is that it is hard to secure an absolutely water-tight structure.

The first concrete elevator was built in 1902 and as soon as the tanks had been filled with grain several of them burst. This naturally retarded the use of concrete for two or three years. However, after one or two concrete elevators had been erected by well-known firms, the elevator owners regained confidence in this material. For some time concrete was used only in foundations and in storage annex, steel or wood being used throughout the working house. Later the storage bins in the working house and the supporting columns were made of concrete, and only the cupola, which is that part of the elevator above the tanks, was built of steel. James Stewart and Company were the first to

introduce a concrete cupola, and the only structural steel now used throughout the house is a very small tonnage required for machinery supports, tension carriages, etc., where concrete would be entirely impracticable. It has been clearly demonstrated that there is no better material for protecting the grain than concrete, because it has a low conductivity for heat or cold, and being denser and freer from porosity than either tile or brick, it

FIG. 3.—BALTIMORE AND OHIO RAILROAD MT. CLARE ELEVATOR, BALTIMORE, MD.

has greater waterproof qualities. Another point in favor of concrete is the low cost of construction, due to the fact that there is practically no skilled labor required in the construction of the modern elevator.

The modern fireproof elevator is so designed as to eliminate the use of wood altogether. The cleaning machines, when they are of wooden construction and in a fireproof house, are completely

steel clad, and the doors must pass the fire underwriters' requirements. No wood is used, even in the office, the housing being of slate enclosed with steel cabinets, which shows how closely the owners follow the requirements of the underwriters. The track shed is equipped with rolling steel doors and the underwriters positively require that no car be allowed to remain in the track shed over night (Fig. 3). The spouting consists of sheet metal and the roof covering generally consists of five-ply tar felt and gravel. All reinforcing steel is medium open-hearth and must conform to the standard specifications. Plain round bars are generally used in the foundations, cupola, etc.; but for the tanks, the horizontal consists of flats, the vertical reinforcement of plain round bars.

METHODS OF CONSTRUCTION.

The equipment for handling concrete in elevator construction differs very little from that used in placing concrete in any other building, excepting in the matter of the forms, in which great advancement has been made in the last few years; the forms for the foundations are stationary, those above the foundation are movable. These forms, made of 2 in. plank, surfaced on one side and two edges, are about 4 ft. 6 in. in height. After the foundation has been completed then forms are set over the whole area and filled with concrete in layers about 8 in. thick; they are raised by a series of jacks, from 6 to 8 on each tank, or if they are used on straight walls they are placed about 5 or 6 ft. apart; these are set in a yoke or framework of steel connected to the wooden forms; through each there is a jacking rod about 1 in. in diameter running vertically (Fig. 4). To operate, a bar is placed in the socket turning a screw, which raises or lowers the jacks on the rod while the forms supported by the jacks remain stationary; by reason of the rod passing through the jack the load is applied concentrically and there is no tendency for the forms to bind. These jacking rods are placed directly on top of each other and no dismantling of the forms is required when additional rods are added. The yokes are connected by means of trusses and these in turn support the temporary floor for the workmen and permits easier handling of material. This continual moving of forms does away with the horizontal rings and discolorations so often seen in

the first concrete elevators. This type of form also has greatly reduced the cost over that with the originally used stationary forms or that with the primitive system first adopted of jacking from the ground all the way to the top of the elevator.

In constructing the working house, the floor girders, where required, are poured simultaneously with the walls, the floor slab generally being put in later, so as not to impede the progress of the wall forms. In reinforcing the tanks, flat bars are placed midway between the forms and at equal intervals, the difference

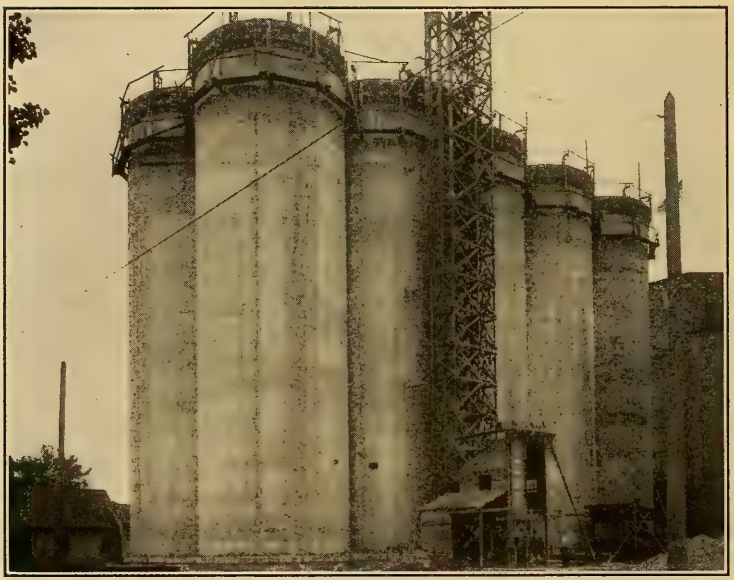


FIG. 4.—NOBLESVILLE MILLING COMPANY ELEVATOR, NOBLESVILLE, IND., IN PROCESS OF CONSTRUCTION.

in pressure below and above being taken care of by the size of the flat. The jacking rod on which the forms are raised is also part of the vertical reinforcing, and similar rods placed between this series of jacking rods form the balance of the vertical reinforcing. The tanks are always laid out in parallel rows. Contacts must be provided for, and this is generally by a system of horizontal anchors and additional concrete in the interstices. This arrangement of bins leaves the space between the different tanks, which is called an interstice bin.

As the elevator is usually placed on the water front and as its boots and receiving pits are necessarily some distance below grade, generally, some provision must be made for waterproofing, which is usually by the membrane system.

The balance of the construction is so nearly allied to other branches of reinforced concrete building that further details are unnecessary.

In conclusion, it is fair to add that the modern elevator has been and will continue to be a great factor in the upbuilding of the great northwest of this country and also of Western Canada; in conjunction with the railroads, the elevator has been responsible for the steady crop increase, until it is safe to predict that in the next half decade the wheat crop of Canada alone will exceed 500,000,000 bushels per annum.

National Association of Cement Users.

HARRISON BUILDING,
PHILADELPHIA, PA.

STANDARD METHODS FOR THE MEASUREMENT OF CONCRETE WORK.

The following divisions are recognized as separate and distinct operations in the construction of concrete work for which separate modes of measurement are necessary:

I. Monolithic Concrete:
- (*a*) Concrete.
- (*b*) Forms.
- (*c*) Reinforcement.
- (*d*) Surface Finish.

II. Sidewalks.

III. Structural Cast Concrete:
- (*a*) Concrete.
- (*b*) Reinforcement.
- (*c*) Erection.

IV. Cast Concrete Trim and Ornamental Work.

The following general rules shall govern the measurement of the above items (with the exceptions where specifically noted):

(*a*) All work shall be measured net as fixed or placed in the structure.

(*b*) In no case shall non-existent material be measured to cover extra labor.

(*c*) No allowance shall be made for waste, voids, or cutting.

I. Monolithic Concrete.

(*a*) *Concrete.*

1. The unit of measure for all concrete shall be the cubic foot.

2. In no case shall the measurement of concrete include the forms.

3. All concrete shall be measured net as placed or poured in the structure.

4. In no case shall an excess measurement of concrete be taken to cover the cost of forms or extra labor in placing.

5. All openings and voids in concrete shall be deducted with the following exceptions:

(*a*) No deduction shall be made for reinforcement, I-beams, bolts, etc., embedded in concrete except where a unit has a sectional area of more than 1 sq. ft.

(*b*) No deduction shall be made for pipes or holes in concrete having a sectional area of less than 1 sq. ft.

(*c*) No deduction shall be made for chamfered, beveled or splayed angles to columns, beams and other work, except where such chamfer, bevel or splay is more than 4 ins. wide measured across the diagonal surface.

6. Each class of concrete having a different proportion of cement, sand or aggregate shall be measured and described separately.

7. Concrete in the different members of a structure shall be measured and described separately according to the accessibility, location or purpose of the work.

8. Concrete with large stones and rocks embedded in same (cyclopean masonry) shall be measured and described according to the richness of the mix and the percentage of rock in same.

9. Concrete in stairs shall be measured by the cubic foot and shall include surface finish when the mixture is the same throughout.

(*b*) *Forms.*

10. The unit of measure for form work shall be the square foot of actual area of the surface of the concrete in contact with the forms or false work.

11. Forms shall in every case be measured and described separately and in no case shall the measurement of concrete include the forms.

12. No deduction shall be made in measurement of surface of concrete supported by forms, because of forms being taken down and re-used two or three times in the course of construction.

13. The unit price for superficial measurement of forms shall include the cost of struts, posts, bracing, bolts, wire ties, oiling, cleaning, and repairing forms. No measurement to be made of these.

14. No distinction shall be made between wood and metal forms.

15. Forms to different parts of a structure shall be measured and described separately according to the position in the structure, accessibility, purpose and character of the work involved.

16. No allowance shall be made for angle fillets or bevels to beams, columns, etc., but curved moldings shall be measured and described separately as hereinafter provided.

17. No deduction in measurement of forms shall be made for openings having an area of less than 25 sq. ft.

18. No deduction shall be made in floor forms for heads of columns of any shape.

19. No deduction shall be made in column and girder forms for ends of girders, cross beams, etc.

20. No allowance shall be made for hand-holes in column forms for clearing out rubbish.

21. The measurement of column forms shall be the girth of the four sides or circumference multiplied by the height from the floor surface to the under side of floor slab above.

22. Forms to octagonal, hexagonal and circular columns shall be measured and described separately from forms to square columns.

23. Caps and bases to columns and other ornamental work shall be measured by number and fully described by overall dimensions.

24. The measurement of beam forms shall be the net length between columns multiplied by the sum of the breadth and twice the depth below the slab, except for beams at edge of floor or around openings which shall have the thickness of floor added to the sum of the breadth and twice the depth.

25. Wall forms shall be measured for both sides of concrete wall.

26. Allowance shall be made by number for pockets left for future beams.

27. Moldings in form work shall be measured by the linear foot.

28. Forms to circular work shall always be measured separately from forms to straight work.

29. No measurement or allowance shall be made for construction joints in slabs, beams or arch ribs, to stop the day's concreting.

30. Construction joints or expansion joints to dams and other large masses of concrete shall be measured by the square foot as they occur.

31. Forms to cornices shall be measured by the lineal foot and the girth stated. (The term girth shall be taken to mean the total width of all curved and straight surfaces touched by the forms.) Plain forms to back of cornice to be measured separately.

32. Forms to window sills, copings and similar work shall be measured by the lineal foot.

33. Forms to the upper side of sloping slabs such as saw-tooth roofs shall be measured whenever the slope of such slab with the horizontal exceeds an angle of 25 degrees.

34. Forms to the under side of stairs shall be measured by the superficial foot.

(*a*) Forms to the front edge of the stairs shall be measured by the lineal foot.

(*b*) Forms to the ends of steps shall be measured by number.

(*c*) *Reinforcement.*

35. The unit of measure of reinforcement shall be the weight in pounds.

36. The weight shall be calculated on the basis of a square rod 1 in. x 1 in. x 12 in., weighing 3.4 lb.

37. Steel rods for reinforcement shall be measured as the net weight placed in the building.

38. Deformed bars shall be measured separately from plain

39. No allowance shall be made for rolling margin.

40. No allowance shall be made for cutting or waste.

41. No allowance shall be made for wire ties, spacers, etc.

42. No separation shall be made according to accessibility, location and purpose of reinforcement except in special cases.

43. In measuring reinforcement the rods shall be measured by the lineal foot as laid. All laps shall be allowed for.

44. The rods of each different size shall be measured and described separately.

45. Bent bars shall be measured separately from straight bars.

46. Pipe sleeves, turnbuckles, clamps, threaded ends, nuts and other forms of mechanical bond shall be measured separately by number and size and allowed for in addition.

47. Wire cloth, expanded metal and other steel fabrics sold in sheets or rolls shall be measured and described by the square foot. The size of mesh and weight per square foot of steel in tension shall be stated. No allowance shall be made for waste, cutting, etc., but all laps shall be measured and allowed for.

(*d*) *Surface Finish.*

48. The unit of measure for finish of concrete surfaces shall be the square foot. Finish shall always be measured and described separately.

49. No measurement or allowance shall be made for going over concrete work after removal of forms and patching up voids and stone pockets, removing fins, etc.

50. Granolithic finish shall be measured by the square foot and shall include all labor and materials for the thickness specified.

51. Finish laid integral with the slab shall be measured separately from finish laid after the slab has set.

52. No allowance shall be made for protection of finish with sawdust, sand or tenting.

53. Grooved surfaces, gutters, curbing, etc., shall be measured separately from plain granolithic and shall be measured by the square foot or lineal foot as the case may require.

54. The following shall be measured by the square foot:

Cement wash. (State how many coats.)
Rubbing with carborundum.
Scrubbing with wire brushes.
Tooling.
Picking.
Plastering.
Etc.

II. Sidewalks and Pavements.

55. Sidewalks and pavements shall be measured by the square foot. The thickness to be stated in the description.

56. Finish, lining in squares and cinder or stone foundations, shall not be separately measured but shall be described.

57. Curbs and curb and gutter work shall be measured by the lineal foot and separated according to character and size, and shall include foundations, forms, finish and cost of special tools if any.

58. In measuring curbs the full height and width or thickness of same shall be taken to the extreme edge.

59. Circular corners to curbs and gutters shall be measured separately by number, stating radius and length measured on the curve.

60. Vault lights shall be measured by the square foot, the size and type of glass, forms, steel and finish to be described but not measured separately. Beams under vault lights shall be measured by the lineal foot. In measuring vault lights the measurement shall go at least 4 in. beyond the outside line of the glass in each direction.

III. Structural Cast Concrete.

(*a*) *Concrete.*

61. The term structural cast concrete is taken to include unit construction by the various systems.

62. The unit of measurement for structural cast concrete shall be the cubic foot, and shall be measured net as provided for monolithic concrete.

63. The various members shall be measured on the ground before erection.

64. No measurement shall be taken of forms.

(*b*) *Reinforcement.*

65. Reinforcement shall be measured separately as provided in Paragraphs 35 to 47, inclusive.

(*c*) *Erection.*

66. The unit of measure for the erection of structural concrete shall be the weight of the finished member in pounds.

67. In measuring the erection of structural cast concrete having a crushed stone or gravel aggregate, the concrete shall be assumed to weigh 150 lb. per cu. ft.

68. No measurement shall be taken of the grouting in structural cast concrete. It shall be deemed to be covered in the price of erection.

IV. Cast Concrete Trim and Ornamental Work.

69. Cast concrete trim shall be measured by the cubic foot, but the measurement shall be the smallest rectangular solid that will contain the piece measured and not its actual content.

70. No allowance shall be made for forms.

71. No allowance shall be made for reinforcement in trim and ornamental work.

72. No allowance shall be made for surface finish in trim and ornamental work.

73. Circular work shall be measured separately from other work.

74. Mitre blocks and end blocks for cornices, etc., shall be measured separately from straight molded work.

75. Vases, seats, pedestals, balusters and other similar items shall be measured by number and description with over-all dimensions.

DISCUSSION.

Mr. Rudolph P. Miller.—Mr. President, might it not be well to provide for cinder concrete, in the same way as stone concrete. Cinder concrete is being used very extensively. Mr. Miller.

Mr. Robert A. Cummings.—I am not aware that cinder concrete is used for structural purposes, and can hardly conceive of an engineer or contractor using such material for that purpose. Hence we did not consider it necessary to provide for it in the rules. It is impractical to prepare rules to cover future possibilities. They should be amended as future conditions require. Mr. Cummings.

Mr. Miller.—Then these rules do not apply to the use of concrete in floor construction in combination with the steel frame structure? Mr. Miller.

Mr. Cummings.—The rules clearly show how concrete is to be measured in such a combination, but not by weight. The weight applies where you use structural concrete in buildings. Mr. Cummings.

National Association of Cement Users.

HARRISON BUILDING,
PHILADELPHIA, PA.

REPORT OF COMMITTEE ON SPECIFICATIONS AND METHODS OF TESTS FOR CONCRETE MATERIALS.

During the past year the work of the Committee has included

(*a*) Layout of tests of concrete for various laboratories with a view to recommending in a later report standards followed in making concrete test pieces.

(*b*) Investigations and compilations of methods of tests of fine aggregate employed by various laboratories.

(*c*) Studies of results of tests of sand mortars with a view to recommending in a subsequent report uniform methods of test of fine aggregates.

The present report emphasizes the necessity for more rigid requirements for the acceptance of concrete materials and for routine tests of the materials and the concrete.

Tests of Concrete.

Tests of the individual materials are not sufficient for determining the quality of the concrete made from them. A certain sand may give satisfactory results with one brand of cement and prove a failure with another. A sand of a certain granulometric composition may require an entirely different proportioning with one coarse aggregate than with another or may be unsuited for use with it because of the sizes of the grains.

The Committee recommends that for all important structures, such as dams, conduits, and buildings, test pieces of concrete be made up in advance of the actual work from materials which it is proposed to use in the work. These test pieces should be made under laboratory conditions so as to obtain uniformity in mixing and storage, and with the same consistency which is to be used in the proposed structure. The test pieces should be stored in moist

atmosphere and compressive tests made at the age of 28 days and also at other periods if practicable. The final selection of the materials and the proportions to be adopted should be governed by the results of these tests.

The tests now in progress in different laboratories under the auspices of the Committee are expected to furnish data from which definite recommendations may be made as to the size and shape of the test piece. The Joint Committee on Concrete and Reinforced Concrete recommend cylinders 8 in. diameter by 16 in. high. Where it is impracticable to adopt this size, a 6-in. cube or a cylinder 6 in. diameter and 6 in. high is tentatively suggested for use with aggregates under $1\frac{1}{2}$ in. in size. It must be borne in mind that test pieces of this shape may give from 10 to 20 per cent higher strength than the same materials made into oblong cylinders or prisms.

Layout of Concrete Tests.—In order to obtain well-defined information with reference to the effect of different shapes and sizes of test pieces and different mixtures, the Committee has laid out in very full detail a number of series of tests. These specifications have been sent to various laboratories throughout the country and accepted by them as a part of their routine tests for the coming year. The results of these tests will be presented in the next report of this Committee.

TESTS OF CONCRETE MATERIALS.

Tests of materials for use in mortar or concrete may be divided into two general classes:

(1) Tests for acceptance.

(2) Tests for quality.

These two divisions frequently merge into each other because in many cases the special characteristics of a sand may render it unfit for certain work, although it may satisfactorily pass a standard test for acceptance.

The most important questions to be asked in regard to a material are: (*a*) Can it be used? (*b*) In what proportions must it be mixed with the other materials to produce the required qualities in mortar or concrete? (*c*) Can another material of the same class be substituted with greater economy?

For small structures and, in fact, frequently for larger ones

where the sources of supply are limited, the all-important question is a combination of the first two, namely, can a certain material—and this applies most frequently to the fine aggregate or sand—be used, and in what proportions to give the desired quality of concrete or mortar? For structures where the volume of the concrete is large, so that the economy of the proportioning is paramount, or for small structures subjected to special conditions—such as the effect of sea water—the cement and aggregates should be thoroughly studied also and special tests made to determine the most economical or the qualifications for the work proposed.

Taking the matter of the selection of sand alone, an illustration of the poor quality that is frequently used in practice is found in the tests in the laboratory of one of the members of the Committee. Out of 37 samples of sand from different parts of the country made into 1 : 3 mortar, 12 were equal to or greater in strength than similar mortar of standard Ottawa sand; 5 were between 90 and 100 per cent of standard sand; 6 were between 80 and 90 per cent; 4 were between 70 and 80 per cent; 6 were between 60 and 70 per cent; 1 between 50 and 60 per cent; 2 between 40 and 50 per cent; and 1 between 30 and 40 per cent. Such results as these indicate the absolute necessity for always testing sand that is used in concrete. For large and important structures, such as dams, tunnels, aqueducts, buildings, and other reinforced concrete work, not only preliminary tests should be made but frequent tests at stated intervals to avoid the danger of inequalities in the supply.

TESTS OF MATERIALS FOR ACCEPTANCE.

The cement shall meet the requirements of the Standard Specifications for Portland Cement of the American Society for Testing Materials and adopted by this Association. (Standard No. 1.)

FINE AGGREGATE.

Sand or other fine aggregate should always be tested. In ordinary cases where special characteristics are not required, the test for acceptance is the determination of the strength of mortar made up with the cement and fine aggregate to be used on the work in comparison with the strength of identical test pieces made

with standard sand. The results of this test also are an aid to the selection of the proportions which should be adopted.

Compression tests of mortar are the most reliable, but tensile tests may be employed when a compression machine is not available.

Selection of Sample.—Take an average sample aggregating not less than 20 lb. Ship in strong box or bag or barrel preferably water tight, so that laboratory will receive material in its natural condition. Ship in a separate package from the cement.

Quartering.—Unless grains are of uniform size, separate from the large sample the quantity required for the test by quartering or other similar means.

Making Test Pieces.—Mold a large enough quantity so that there will be at least four test pieces each to test at 72 hours, 7 days, and 28 days, in proportions 1 part cement to 3 parts sand by dry weight. Bank sand should not be dried, since drying may affect the quality, but the percentage of moisture should be determined by test on another sample, and allowed for in making the weights. The consistency should be the same as that required for standard sand mortar. From 10 to 40 per cent more water may be required for bank sands or artificial aggregates than for standard Ottawa sand to produce the same consistency.

Similar test pieces of the same cement mixed at the same time under the same conditions should be made with standard Ottawa sand in the same proportions.

All temperatures of mixing and storage should be maintained at 70° F. During the first 24 hours the test pieces should be stored in moist air and the remainder of the period in water.

The 72-hour test is the most severe, and sand failing to attain the requirement at this age frequently reaches it at 7 or 28 days and can be accepted. If, however, the 72-hour briquets break in the clips of the machine or if the test pieces at this age show very low strength, say 25 per cent, or below, of the strength of standard sand mortar, the sand should be considered dangerous to use on any important work of construction.

Requirements for Strength.—The aim in concrete construction should be to obtain a sand which produces a 1 : 3 mortar equal in strength to a similar mortar of standard Ottawa sand. Sands producing mortars having a strength of less than 70 per cent of standard sand mortar should be rejected.

Between these two limits the variations in strength should be taken into account in fixing the proportions of the mortar or concrete.

Other Tests.—For work requiring special characteristics, the fine aggregate must be accepted not merely on passing the tests for strength, but other tests, such as the test for permeability, the effect of different brands of cement, the effect of frost action and the effect of fire. Characteristics of yield, density, chemical composition, granulometric composition, and amount of organic matter, are frequently required.

COARSE AGGREGATES.

It is recommended that the quality of the coarse aggregate be determined from the preliminary tests of concrete made with the materials and in the proportions designed to be used in the work. Government tests made at the Structural Mate ials Laboratory in St. Louis under the direction of Mr. Richard L. Humphrey, clearly show that the hardness of the particles of coarse aggregate appreciably affect the strength of the aggregate, so that greater stresses may be used in concrete when the stone is hard than when it is soft. In certain cases the chemical composition of the coarse aggregate may affect the strength and durability of the concrete.

TESTS FOR QUALITY OF FINE AGGREGATE.

In addition to the tests for acceptance which have been discussed in the preceding paragraphs, many aggregates require special tests. Furthermore, various laboratories have adopted different classes of tests to use with aggregates which come to them for examination. This can be discussed best by a consideration of the practice in different laboratories.

During the past year this committee has been in communication with all of the laboratories of which they were able to learn, in an effort to gather as complete data as possible relative to the various tests for a sand now in use. Each laboratory was requested to furnish a complete description of their methods of testing sands—in most cases such descriptions were freely furnished. In all, 28 laboratories were communicated with and replies were

received from 24; of these, 15 gave more or less complete descriptions of their tests.

The data in these replies were tabulated and listed and gave some very interesting information. It was found that some 26 different steps are taken in these laboratories for the determination of the suitability of a sand for use in concrete or mortar.

Listed and briefly described, these steps are as follows:

1. *Examination of Deposit* to determine its uniformity, stratification, extent, overburden, methods of stripping, handling, screening, washing, etc.

2. *Preparation of Sample* to eliminate stones, water, silt, and to insure its being representative of the bank or the quantity from which it was taken.

3. *Tensile Test of Mortar* to determine the strength of known mixtures of the sand and Portland cement at various ages and to compare this strength with that of standard Ottawa sand.

4. *Compressive Test of Mortar* to determine the strength of known mixtures of the sand and Portland cement at various ages and to ascertain what mixture is necessary to produce a required strength, or impermeability.

5. *Compressive Test of Concrete* to determine the strength of known mixtures of the sand and a coarse aggregate and Portland cement at various ages and to ascertain what mixture is necessary to produce a required strength or impermeability. In the case of bank gravel or crushed stone which is not to be screened and reproportioned this test is used instead of No. 4 above.

6. *Percentage of Moisture* is determined in order to allow for the water contained in the sand in proportioning mixtures, computing freight rates, etc.

7. *Percentage of Voids* is determined:

(*a*) By determining the specific gravity of the solid particles and weighing a known volume of the sand and computing therefrom the precentage of voids.

(*b*) By filling the voids in a known volume of sand with a measured amount of a liquid.*

8. *Yield, or Volume of Mixture* is determined by adding a known quantity of cement to a known quantity of the sand,

* This method as usually practiced is very inexact on account of the air entrained.

wetting the mixture and noting the increase in the volume of the sand. This is repeated with various proportions of cement, and sand to ascertain the amount of cement required to fill the voids in the sand.

9. *Density of Mortar and Concrete* is determined for various mixtures of cement and sand and of cement, sand and stone to ascertain that mixture which gives a material of the greatest weight per unit of volume.

10. *Specific Gravity of Solid Particles* is determined by three different methods:

(*a*) By the specific gravity apparatus such as the Jackson flask.

(*b*) By pouring a known weight of sand into a known volume of water and noting increase of volume of liquid.

(*c*) For coarse aggregate by suspending pieces by a thread on chemical scales and noting weight in air and weight when hanging in a jar of water.

11. *Specific Gravity of Sand* including voids is determined by weighing a known volume of the sand exclusive of moisture and computing the specific gravity from the known weight of the same volume of water. Having determined the specific gravity of the solid particles (Test 10) and also that of the sand (including voids) the per cent of voids in the sand may be calculated.

12. *Weight per Cubic Foot* is determined by weighing a known volume of the sand. This test is very closely related to Test II.

13. *Granulometric Analysis or Mechanical Analysis* is made by passing a known weight of the sand through a series of sieves of various sizes and noting the amount retained on each sieve and the amount passing the finest one. This test shows the distribution of the particles from fine to coarse.

14. *Uniformity Coefficient* is determined from a curve plotted from the granulometric analysis by dividing the diameter of the particles at the point where the curve crosses the 60 per cent ordinate by the diameter of the particles at the point where the curve crosses the 10 per cent ordinate.

15. *Wet Screening* of a sand on the 100 and 200 mesh sieves shows the amount of fine material which can be washed from the sand and which in the case of dry screening in Test 13 often adheres to the larger particles and so leads to uncertain results.

16. *Silt Suspended in Water* is determined by agitating a quantity of the sand with a large excess of water in a tall cylindrical glass vessel and noting the amount of silt which becomes suspended in the water and later settles down on the top of the sand.

17. *Silt Washed Out* is found by agitating a known quantity of the sand in water, allowing the coarse particles to settle and decanting or syphoning off the muddy water from the top. This operation is repeated until clear water is obtained. The amount of silt washed out is determined after evaporating the water.

18. *Loss on Ignition* is determined by heating to a red heat a weighed quantity of the dried sand and again weighing after cooling. This test is best performed only on the silt washed from a quantity of sand as the silt usually contains the injurious organic constitutents of the sand.

19. *Organic Matter* is sometimes determined after the methods employed by the agricultural chemists.

20. *Chemical Analysis* is made to determine the character of the sand grains. Only a few of the most common rock constituents are usually determined quantitatively, for the purpose of judging of the strength and durability of concrete made from the sand. A high silica content is usually desired.

21. *Microscopical Examination* is made to ascertain the approximate size of the grains, their shape, character of surface and to detect the pressure of objectionable material or foreign matter such as mica or small roots or a coating on the grains.

22. *Absorption of Mortar and Concrete* made from the sand mixed in varying proportions with cement or cement and stone is determined to ascertain its suitability and the proper proportions for the production of a product having a minimum of absorption.

23. *Permeability of Mortar and Concrete* made from the sand is sometimes determined for various proportions of the constituents in order to determine mixtures for the production of an impermeable mortar or concrete.

24. *Effect of Different Cements* on some sands is quite variable and tests are sometimes made to determine with which of several available cements the sand will give best results.

25. *Freezing* during the setting time has a more injurious effect on mortar and concrete made with some sands than those

made by others and tests are made to determine the extent to which the sand is affected by freezing.

26. *Fire* has a more disastrous effect on some sands when used in mortar or concrete than it has upon others and a heat test is made to show the ability of the sand to resist fire.

NUMBER OF LABORATORIES USING EACH TEST.

TEST.	NUMBER OF LABORATORIES.
1. Examination of Deposit	4
2. Preparation of Sample	8
3. Tensile Test of Mortar	15
4. Compressive Test of Mortar	7
5. Compressive Test of Concrete	5
6. Per cent of Moisture	7
7. Per cent of Voids	15
8. Yield or Volume of Mixture	3
9. Density of Mortar and Concrete	2
10. Specific Gravity of Solid Particles	4
11. Specific Gravity of Sand	5
12. Weight per Cubic Foot	2
13. Granulometric Analysis	16
14. Uniformity Coefficient	4
15. Wet Screening	1
16. Silt Suspended in Water	4
17. Silt Washed Out	8
18. Loss on Ignition	5
19. Organic Matter by Agricultural Methods	1
20. Chemical Analysis	5
21. Microscopical Examination	5
22. Absorption	1
23. Permeability	3
24. Effect of Different Cements	2
25. Freezing	1
26. Fire	1

No doubt other tests of fine aggregate are in use which have not come to the notice of the Committee, but the very large range and lack of uniformity is illustrated by the outline given. No two of the laboratories used the same tests, and scarcely two of them carry out their tests in exactly the same manner, so that at present it is difficult or impossible to accurately compare the results obtained in one laboratory with those obtained in another.

Examination of the list shows that the popular tests for sand

at the present time are granulometric analysis, tensile tests of mortar, and percentage of voids. Of these only one, the tensile test of mortar, belongs properly in the classification of tests for acceptance as discussed in preceding paragraphs of the report.

The problem before the engineer who is interested in proper methods of testing materials is to select and standardize such tests as will give the necessary data which will determine whether or not the sand is suitable for a particular use and, if suitable, in what proportions it should be used to produce desired qualities in the resulting mortar or concrete.

In the further study of this problem the Committee desires to enlist the active co-operation of all those who are interested in the subject, and shall appreciate any information or the results of any tests which may lead toward the solution.

Respectfully submitted,

SANFORD E. THOMPSON,
Chairman.
CLOYD M. CHAPMAN,
Secretary.
WILLIAM B. FULLER,
RUSSELL S. GREENMAN,
ARTHUR N. TALBOT.

DISCUSSION.

Mr. Thompson.

MR. SANFORD E. THOMPSON.—I wish to make a few further remarks with reference to the concrete tests referred to in the report. We are starting in with a plan which is somewhat different from that usually employed in committee work in order to obtain actual information based on comparative tests. Your Committee laid out five series of tests, and instead of sending out simply the outline of these, it drew up full detail specifications, covering in all some ten pages, telling exactly how the work should be carried on. After corresponding with a number of laboratories (most of them college laboratories), nine agreed to take up the tests, these including such universities as Illinois, Indiana, Wisconsin, California and Massachusetts Institute of Technology.

You may say that college tests, especially if made by students, frequently give results which are unreliable. In the present case, however, the most of the universities engaged in the work have had considerable experience in testing concrete and already have made researches of value. Furthermore, the results aimed at are relative, so that if each college follows uniform conditions throughout its tests, these conditions need not necessarily exactly coincide with the methods employed by others. For example, one of the series of tests proposes to make up sets of cubes of sizes ranging from 4 in. to 12 in. on a side, for the purpose of determining whether the size affects the homogeneity sufficiently to give a variation in strength. Theoretically, the different sized blocks should give the same strength per square inch, but we do not know definitely whether this is true in practice.

Another comparison will be between cubes and cylinders. Another comparison will be between prisms of different heights ranging from two inches in thickness up to six times the least diameter. From the results of these, we should be able to more accurately compare the strength of, say, an 8 x 16-in. cylinder with an 8 x 8-in. cube, or an 8 x 8-in. cylinder.

Mr. Thompson.

Before we can recommend definite sizes to be used for concrete specimens, we must have data obtained from tests of this kind. It is odd that such simple comparative tests have never been made before to an extent which would make the results reliable. It is probably because of their very simplicity. Laboratories and men making experiments like to produce results which will be out of the ordinary. They frequently overlook the simpler tests which tend to give results of more value, although less striking.

One of the series compares the strength of concrete of different consistency. The making of the specifications for this involved a description of the method of fixing a uniform consistency. A method of test was finally established based on a suggestion of the Secretary of our Committee. A cone about 4 in. in diameter at the base and 6 in. high is made and a sample mixture of concrete is placed in it. A glass plate is placed on top of the concrete and the cone is inverted so that the point is at the top. The cone is then removed. If the concrete slumps, it is too wet. If it stands perfectly rigid, it is too dry. When the concrete is just wet enough so as to begin to change its shape and yet not actually slump down, it is of proper consistency for the test.

During the coming year, if the Committee is continued it proposes to keep in touch with the various laboratories that are making these tests, and after the tests are completed, to have them tabulated and determine what conclusions can be drawn from them. The Committee proposes further to continue its investigations and tests of sand, which is probably the most important question of testing that the concrete engineer has to deal with.

Any information or assistance that can be given us by members of the Association will be gladly accepted.

THE CONSTITUTION OF PORTLAND CEMENT. SOME RESULTS OBTAINED AT THE EXPERIMENTAL CEMENT PLANT OF THE BUREAU OF STANDARDS.

By P. H. Bates.*

The constitution of Portland cement is at least a matter of considerable interest, if not of fundamental importance for its proper use. Notwithstanding the fact that this problem has been worked upon for a great number of years, yet it must be acknowledged that agreement among investigators is lacking to a great degree. The point of contention is whether the three essential ingredients—silica, alumina and lime—are present as a ternary compound (one containing three radicles) or as a binary radicle (one containing two radicles). The adherents of the former theory, consider that the silica, alumina and lime are combined to form a compound which is given as having a different composition by different investigators. This is the material to which the name "Alite" is given. Those who support the idea of binary compounds alone, if they recognize the material "Alite," hold that it is either tri-calcium silicate or a solid solution of this silicate in tri-calcium aluminate. The presence of the ortho-silicate (di-calcium silicate) as well as a di-calcium aluminate has been generally acknowledged. These, when more or less colored with iron, have been referred to as Belite, Celite or Felite. It might be remarked that the optical data, which has been given by many investigators regarding these latter, is seldom sufficient to identify them, so that it frequently happens that the Belite of one is not the Belite of another. As the problem now stands, the question hinges upon whether the so-called Alite contains alumina or is only tri-calcium silicate. The possibility of there being a compound of the formula of $3CaO.SiO_2$ in Portland cement has been a matter of discussion for a great many years. In this country Newberry and Richardson have found present in the material which they

* Chemist, Bureau of Standards, Pittsburgh, Pa.

made a compound which they claimed to have this formula, although this is contradictory to the results of nearly all foreign investigators with few exceptions, though among the latter was LeChatelier. The early work of the Geophysical Laboratory of the Carnegie Institution also pointed to its non-existence, but the last work of this laboratory, but comparatively recently published, acknowledged that there is a possibility of this compound being present in cement. However, they were not able to find it present in the materials made and examined by Newberry and Richardson. According to the recent work done by foreign investigators, Kaiserman* has observed its presence, while Janecki† denies not only its presence, but, referring to the work of the Geophysical Laboratory, doubts the possibility of there being a binary compound under such conditions.

There has been installed at the Pittsburgh Branch of this Bureau a 2-ft. x 20-ft. rotary kiln with the necessary grinding apparatus, for the study of Portland cement. It was recognized from the first, that in order that the studies, which were to be made, might be complete, a petrographer with a complete outfit for petrographic studies must be installed All burns would then be examined for their constitution according to the most approved and exacting methods. These notes therefore are a discussion of the qualitative constitution of the first burns made in this kiln.

It was thought advisable to publish some notes on the constitution of cement at present, in view of the fact that a complete discussion of all the work done would of necessity not be made until the results of the physical tests of the cement were available; as specimens were made up for two-year periods, this would put off publication to a remote date. Also, there has not been developed a sufficiently accurate method to determine quantitatively the amounts of the various constituents present. There are several difficulties in the employment of thin sections of clinker in order to determine quantitatively the constituents. The first reason is the great difficulty to avoid overlapping grains. The sections, owing to the nature of the clinker, cannot be made less than 0.03 millimeter in thickness. The grains present in the clinker are

* Constitution of Cement, Kaiserman, *Cement & Eng. News*, Jan. and Feb., 1911. *Jour. Soc. Chem. Ind.*, Dec. 15, 1910.

† Constitution of Portland Cement, Janecki. *Zeit. Anorganische Chemie*, Vol. 73, Part 2.

usually smaller than this, consequently, it happens in the section that one grain may be on top of another or may partly overlap another, thus destroying its distinguishing optical properties. It is also impossible to determine more than approximately the index of refraction of a constituent present in a section. This is one of the most important data which is necessary for recognizing individual constituents or crystals. The development of a method to secure accurate quantitative results promises to be a matter of some time.

In view of the greatly superior quality of the work of the Geophysical Laboratory with compounds made from pure materials, and of the accurate and detailed optical constants which they have determined for these, their methods, data, and results have been used by this laboratory as a standard. It must also be mentioned that burns made on pure materials in this laboratory

TABLE I.

	14	2A	13	15	7	10	9	11
SiO_2	27.10	26.32	25.00	23.40	22.13	20.11	18.44	18.71
Al_2O_3 } Fe_2O_3 }	11.52	11.83	11.92	11.32	10.72	12.37	11.50	12.24
CaO } MgO }	59.84	61.36	62.29	64.32	66.35	67.29	67.69	68.65

have checked the results which they obtained. It has, therefore, been assumed from these results that commercial Portland cement may contain tri-calcium silicate, di-calcium silicate, calcium aluminates, in which the lime and alumina are combined in the ratios of either 3 to 1, 5 to 3, or 1 to 1, and free lime. Thus the presence of a low alumina ternary compound and a di-calcium aluminate is not recognized.

In the first series of burns made, the alumina and iron oxide content has been kept approximately constant, while the silica and lime content have varied—from a low silica and high lime content to a high silica low lime content. The limit in each case has been the limiting product which could be obtained in the rotary kiln. Thus a higher silica content would have produced a material which would have been in such a plastic state in the kiln that such large balls of soft clinker would have formed as to so materially reduce the draft that proper combustion would

have been prevented; low temperature in this case would also have produced only dusting clinker. A higher lime content burn could not have been burned, since a sufficiently high temperature for such a burn could not be obtained at present.

The partial analysis of the burns is given in Table I. The iron oxide and alumina are combined (the iron oxide averaging 3.50 per cent) as well as the lime and magnesia (the magnesia content averaging 1.70 per cent).

In Table II it is noticed that free *CaO* is usually present;

TABLE II.—PETROGRAPHIC EXAMINATION OF SOME BURNS OF SERIES I.

Burn No.	Free *CaO*.	Beta *2CaO.SiO₂*.	*3CaO.SiO₂*.	*5CaO.3Al₂O₃*.	*3CaO.Al₂O₃*.	Iron Oxide.
14	None.	Major constituent.	A little.	More than the 3–1. Also some *CaO*, *Al₂O₃* noted.	A little.	As *Fe₃O₄*, as a "ferrite" and coloring *2CaO.SiO₂* and 5 : 3 alumina.
2A	Very little.	Major constituent.	A little.	The aluminates are present in about equal quantities.		Coloring ortho-silicate and glass and 5 : 3 aluminate.
13	Very little.	Major constituent.	A little.	A few grains.	Considerable.	*Fe₃O₄* and coloring *2CaO.SiO₂*, and glass.
15	Very little.	Major constituent.	More than above burns.	A little.	Considerable.	*Fe₃O₄*, coloring *2CaO.SiO₂* and glass.
7	More than above burns.	Major constituent.	A few grains.	A little.	Considerable.	*Fe₃O₄*, "ferrite" and coloring *2CaO.SiO₂*.
10	Very little.	Major constituent.	Considerable.		Only aluminate.	"Ferrite" and coloring *2CaO.SiO₂*.
9	Considerable.	Major constituent.	Considerable.		Only aluminate.	Appears only as a "ferrite."
11	Considerable.	Major constituent.	Considerable.		Only aluminate.	*Fe₃O₄*, coloring *2CaO.SiO₂* and a "ferrite."

but except in Numbers 9 and 11, the pats made of these cements were perfect in both water and steam, even though the clinker was ground immediately after burning and the ground cement not further aged. The major constituent is the beta ortho-silicate; in 2 burns (Nos. 10 and 11) there was apparently some alpha ortho-silicate, but this was in but minute quantities, and may indeed have been the beta, as there is much difficulty in distinguishing between these two. As these examinations are of the clinker, no gamma ortho-silicate was found, since this is the form of the silicate which is produced when clinker dusts. Burns 14, 2-A

and 13, however, did dust to a slight extent, and the gamma silicate was then present in the dust. The tri-silicate was always present, in the low silica burns in but minute quantities, in the higher silica and higher burning temperature burns in much larger quantities.

Mono-calcium aluminate was found in one of the cements. In the two of highest silica content, it is noticed that there was both the 5 to 3 and the 3 to 1 aluminate, about equal quantities, but with increasing silica content the 5 to 3 aluminate decreases until in the burns containing under 20 per cent silica, it is absent. It should be borne in mind however that the question of temperature of burning enters into consideration here; the low silica burns were made at higher temperatures than the high silica ones, and with increasing temperatures, there is a tendency for the formation of increasing amounts of the 3 to 1 aluminate.

It has been found almost invariably that the iron oxide exists in three different forms. No matter whether the clinker be well-burned or underburned, or whether it contains a high or low percentage of lime, the iron oxide always enters into the beta-ortho-silicate. The grains of the latter are colored brownish to red. When the 5 to 3 lime aluminate is present, it also colors it, but it has not been noticed to color either the tri-silicate or tri-aluminate. It also occurs in the interstices between the other cement minerals. In the case of low lime cements, this interstitial material is double refracting, has an index of refraction greater than 1.75, and seems to correspond optically to the compound $2CaO.Fe_2O_3$. With the very high lime content cements and higher burned cements, this compound disappears and instead there is formed an almost opaque glass and a small amount of magnetite.

In general it is also noticed that those constituents are present, as would be expected from the crystallization curve as outlined by Shepherd and Rankin.* In no case was there observed a ternary compound of the composition $2CaO.Al_2Oi.S_3O_2$. It might also be remarked that the tri-calcium silicate which has been insisted upon by many foreign investigators as containing alumina, has been tested for this impurity by means of Patent Blue. This dye has been found in this laboratory to stain com-

* A preliminary report on the ternary system $CaO.Al_2O_3.SiO_2$. A study of the Constitution of Portland Cement Clinker, Shepherd & Rankin. *Jour. Ind. & Eng. Chemistry*, Vol. 3, No. 4.

pounds containing alumina (previously noted by Kaiserman), and in no case has it been found that the compound called tri-calcium silicate was stained. It therefore did not contain alumina.

It was mentioned above that burn 14 dusted. This dusting, however, was different from that usually obtained or noticed with cement clinker, in that it did not start within a very short time after leaving the kiln and also that the dusting was not immediately to an extremely fine powder. In this particular case in about two hours after the clinker dropped from the kiln and when it had fallen to about room temperature, some of it disintegrated into pieces about ½ in. in diameter. These pieces then broke up into still smaller ones, and this breaking up was continued until a fine powder was reached. Not all of the clinker dusted however. On examination of these larger pieces at the end of several days, it was found that beta prime (β') ortho-silicate was present. In the dust, however, it was absent and had apparently changed into the gamma ortho-silicate. This is the first case of which there is any record of the beta prime silicate changing into the gamma. The usual dusting of cement is the change of the alpha or beta into the gamma, but, as noted above, the mechanical operation of this change is different from the change of the beta prime into the gamma.

An approximate quantitative determination of the constituents in the clinker was made of a number of thin sections prepared from them. The method consists of observations with the highest magnification, using a Ramsden positive ocular and plate holder in which is inserted a double rectangular co-ordinate plate. This plate consists of two series of parallel lines at right angles to each other, the lines being equally spaced. This gives rise to a field made up of a series of squares. The constituents present in each square are estimated quantitatively, the unit being a square 0.01 mm. on a side. At least twenty fields are taken throughout the section and estimated, and the average represents the approximate quantitative composition. The ortho-silicate was estimated alone, the tri-silicate and aluminates together, and the iron compounds other than those coloring ortho-silicates; and are given in this manner in the table. As mentioned before, in thin sections it is extremely difficult to distinguish between the aluminates and tri-calcium silicates. Table III gives the results on this series of burns.

TABLE III.

Burn No.	Per cent.		
	$2CaO.SiO_2$.	Aluminates and $3CaO.SiO_2$.	Iron Compounds.
14	66.0	11.1	22.2
2A	61.4	31.4	7.0
13	58.1	33.0	8.75
15	46.0	45.6	8.2
7	45.0	44.3	10.6
10	40.2	51.4	6.4
9	31.7	59.9	6.3
11	28.3	62.2	8.2

TABLE IV.

	Burn No.					
	20	19	18	15	23	17
SiO_2	23.42	23.97	24.11	23.40	23.69	23.57
Al_2O_3	15.92	14.01	12.41	11.32	9.35	8.07
Fe_2O_3						
CaO	60.66	60.89	62.89	64.32	66.95	68.25
MgO						

The quantitative petrographic examination is given in Table V.

TABLE V.

Burn No.	Free CaO.	Beta $2CaO.SiO_2$.	$3CaO.SiO_2$.	$5CaO.3Al_2O_3$.	$3CaO.Al_2O_3$.	Iron Oxide.
20	None.	Major constituent.*	None.	Much.†	None.	Colors Beta ortho-silicate, 5:3 and 1:1 aluminate also as a ferrite, no Fe_3O_4.
19	Very little.	Major constituent.	A little.	A little.	Much more than $5CaO$, $3Al_2O_3$.	Coloring orthosilicate as a glass and Fe_3O_4.
18	Very little.	Major constituent.	A little.	A little.	Much more than $3CaO$. $3Al_2O_3$.	Coloring orthosilicate and as a glass—a little Fe_3O_4.
15	Very little.	Major constituent.	More than above burns.	A little.	Much more than $5CaO$. $3Al_2O_3$.	Coloring orthosilicate and as a glass and Fe_3O_4.
23	Some.	Equal quantities			Only aluminate present.	Coloring orthosilicate as a glass and Fe_3O_4.
17	Some.	Equal quantities			Only aluminate present.	Coloring orthosilicate, Fe_3O_4 and as a glass.

* B′ $2CaO.SiO_2$ present.
† Some $CaO.Al_2O_3$ present.

Owing to the lack of ability to distinguish between the tricalcium silicate and the aluminates, not much of value is obtained

from Table III. But it is noticeable that in a decrease of about 7.5 per cent silica, and an increase of about 9.5 per cent lime, there has been a decrease of approximately 38 per cent ortho-silicate and an increase of approximately 50 per cent combined aluminate and tri-calcium silicate. As there was but little increase in the alumina content, it can be seen that there has been a very large increase in the tri-calcium silicate content.

A second series of burns has been made in which the silica content has been kept constant, and the alumina-iron oxide, and lime content varied. The partial analyses of the members of this burn are given in Table IV.

This series shows about the same results as the first one. While free lime was noticed in all but one case, yet this was in very small quantities and in every case the pats, both in steam and

TABLE VI.

Burn No.	Per cent.		
	$2CaO.SiO_2$.	Aluminates and $3CaO.SiO_2$.	Iron Compounds.
20	63.55	17.3	19.2
19	60.8	30.9	8.3
18	49.45	39.65	10.8
15	46.0	45.6	8.2
23	26.5	62.6	10.5
17	25.7	64.2	8.5

boiling water, were satisfactory. The variation of the other constituents takes place in the same manner as before—with increasing lime content, there is an increase in the amount of tri-calcium silicate and tri-calcium aluminate. In the case of the latter, this increase takes place, also when the amount of alumina present decreases. However, in this latter case, the lime content has been increased and also the temperature of burning. Beta prime ortho-silicate was again noted in burn 20 which dusted in a manner similar to burn 14.

An approximate quantitative examination of thin sections of the clinker of this series, made in a similar manner as in the previous series gave the results shown in Table VI.

The very large increase of the percentage of aluminate and tri-calcium silicate is here immediately apparent. By referring

to the chemical analyses, it will be seen that burn 20 contains the greater amount of iron-alumina and least lime, whereas, burn 17 contains the least iron-alumina and greater amount of lime. In all the alumina of burn 19 (actually present 9.39) were calculated to tri-calcium aluminate and this deducted from the total percentage of this aluminate and tri-calcium silicate as given above there would remain 15.1 of the latter. (Note that in burn 20 no tri-calcium aluminate was present.) Making the same calculation in burn 17, where the alumina actually present is 5.78, we find that there is approximately present 54.7 per cent of tri-calcium silicate. While this amount of this latter silicate plus the amount of ortho-silicate found would require considerably more silica than was actually present in the clinker, yet there must be borne in mind the difficulties and inaccuracies in the quantitative petrographic examination of thin sections. The greater increase in tri-silicate, however, remains clearly evident.

No attempt has been made at present to correlate, this change in constitution with the physical properties of the finished cement. Such a discussion is reserved for a later period, when, as stated before, a more accurate method of quantitative examination will have been developed, and also when more burns will have been made. Attention must be called especially to the fact stated before, that the changes noted above in constitution, have been brought about not only by changes in composition but by changes in temperature also. In any case, that temperature was used which would give a clinker, the ground cement of which would satisfactorily pass the standard specifications. It was necessary of course to increase the temperature as the lime content was increased. A further study will consist of noting the change of constitution of burns made from various raw mixes as the temperature is changed. This is a matter of considerable time, as a number of burns and examinations will have to be made of each mix. Another study outlined is the accurate determination of all the iron compounds present. As noted above, the iron colors the ortho-silicate and 5 : 3 aluminate, but does not otherwise materially change the optical properties of these. A study of the relation of the iron to these compounds will be made in a manner similar to that made by Klein and Phillips* of this laboratory in their

* Magnesia in Portland Cement, Klein and Phillips, Eighth Inter. Congress of Applied Chemistry, Section IIIc.

work on the relation of magnesia to the ortho-silicate and tri-calcium aluminate. At the same time the lime ferrate sometimes found can also be studied more closely.

In conclusion particular credit must be given to Mr. A. A. Klein, of this laboratory, who has made all thin sections and the petrographic analyses.

DISCUSSION

Mr. Boynton. MR. C. W. BOYNTON.—I would not undertake to discuss this splendid paper in its entirety, but in the early part reference was made to burning clinker at a much lower temperature than we are in the habit of assuming is used in ordinary practice. I presume Mr. Bates measured the heat obtained in the small kiln used in making these experiments, and I wish to ask him how the temperature in the small kiln would, in his opinion, compare with the heat ordinarily obtained in practice.

Mr. Bates. MR. P. H. BATES.—I think the temperatures are about the same, but we naturally assume that those in the rotary kiln are much higher than they are. For instance, it is usually reported that the Germans burn cement at 1,600° C. I have had the pleasure of talking with a number of German ceramists during the past summer, and asked them, after they had gone through a number of our cement plants, if, so far as they could judge, the temperature of our kilns was higher or lower than theirs. They seemed to think that the temperature of our kilns was somewhat higher, and they hardly believed it was possible for them to obtain quite as high temperatures as we do on account of the inferior quality of their coal. The highest temperature which we have used in making cement in the experimental kiln was 1,538° C. and that burns a very high lime cement satisfactorily. We have burned high silica and high alumina cements at temperatures as low as 1,350° C. That temperature compares very favorably with that obtained in ordinary practice. I would even say that I think it is somewhat higher. The determination of temperature in a rotary kiln is a very difficult matter, and when attempted is usually carried on in a very crude manner by some form of radiation pyrometer, the use of which is always open to considerable criticism and to a very large correction in order to express the temperature according to "black body" conditions under which the pyrometer has been calibrated. While we do use such a type of pyrometer—Fery—to control our temperatures, we do not use it for the actual measurement. This latter is obtained by the use of a thermocouple.

Mr. Boynton.—What heat is required in burning commercial mixtures? Mr. Boynton.

Mr. Bates.—Between 1,400° and 1,500° C. Our mixes are all commercial. We use clay obtained from Ohio and limestone from York, Pa. The latter is the same stone used by one of the manufacturers of white Portland cement. It is essential, however, when the composition is changed, to add other materials; as, for instance, to increase the silica content, we add either powdered sand or ground flint. To increase the alumina content we add kaolin. We have, moreover, found no difficulty in having powdered sand combine with the lime, as we have proven conclusively by examining the burnt cement for insoluble residue. In ordinary practice the presence of sand or quartz in the clay must be guarded against solely for the inability to grind this material. If raw materials high in quartz were examined after grinding, it would be found that the coarse material is almost entirely quartz. If this quartz were reduced to the same degree of fineness as the rest of the material, and incorporated with the raw mix, it would be found after burning that it had combined completely. We grind all mixtures to a fineness of 85 per cent through the No. 200 sieve. Mr. Bates.

THE PROGRESS OF THE WORK OF THE BUREAU OF STANDARDS IN CEMENT AND CONCRETE.

By Rudolph J. Wig.*

The National Bureau of Standards was established by Congress in 1901 with broad functions which included not only the development and custody of the standards of weight and length, but the standardization of measuring apparatus, the determination of physical constants, and the determination of the properties of materials for general use in technology and trade. Every industry has its units for determining both quantity and quality, with particular methods of measurement requiring its special measuring instruments.

Copies of the international weights and measures are in the possession of the Bureau of Standards from which all other standards of weight and length used in the United States are derived. Measures of weight, length, and volume, such as gages, bars, rules, tapes, screens, sieves, scales, weights, and all classes of volumetric apparatus are standardized for the public under definite conditions, and certificates of accuracy are issued. Researches are in progress relating to the construction and maintenance of standards of different types.

While the value of much of this work is observable only indirectly in the cement industry, much of it is basic in the standardization of weighing scales, weights, sieves, chemical apparatus, specific gravity bottles, etc., and in affecting all the units of length, weight and volume used by the engineer and the scientist. While it may not be that any of the apparatus which he uses has actually been calibrated by the Bureau, the effect of the Bureau's efforts in standardization of apparatus and instruments has been to encourage greater precision and accuracy in their manufacture.

Similar work is being done by the Bureau in heat and thermometry, chemistry, photometry, barometry, chronometry, optics, and electricity, establishing units of standard, investigating the design, behavior and accuracy of apparatus with the improvement of methods in manufacturing and testing.

* Associate Engineer, Bureau of Standards, Washington, D. C.

One of the important functions of the Bureau is the routine testing of materials, principally on behalf of the Government, which includes tests of cements, steels, alloys, bricks, clays, limes, non-ferrous metals, rubbers for various purposes, leathers, papers, cloths, inks, paints, twines, ropes, oils, and similar miscellaneous materials. In conjunction with these routine tests studies are made of the practical and scientific basis for specifications, the desirable qualities in materials, their accurate description in terms of physical and chemical properties which may be measured by standard tests or analyses, standard methods of sampling, standard instruments and methods of testing, and finally the preparation of standard specifications for the guidance of the manufacturer and purchaser. This work should eventually result in the adoption of definite specifications and tests for all materials in use and thus furnish a scientific basis for manufacture and sale and a reliable and impersonal standard for inspection and acceptance.

Prior to July 1, 1910, the Bureau of Standards had done very little research work in cement, but conducted a small routine cement testing laboratory, established in 1903, in which material was tested for the public and some of the Government offices in Washington. Most of the cement testing at this time was done either in the individual departments or by the Technologic Branch of the United States Geological Survey. The structural materials investigations were transferred by act of Congress from the United States Geological Survey to the Bureau of Standards. This included the transfer of four laboratories which were located in Pittsburgh, Pa., Northampton, Pa., Atlantic City, N. J., and Washington, D. C., with equipment and accumulated data. With the exception of the Atlantic City laboratory practically all the work of the other laboratories at this time was routine, and was not interrupted during the transfer.

The accumulated data transferred and their condition at the time of transfer were as follows:

Reinforced Concrete Beams.

Series 2.

Tests of 333 reinforced concrete beams, with cylindrical and cubical compression test pieces. These were made as part of the

general program, as outlined by the Joint Committee on Concrete and Reinforced Concrete, published in Bulletin 329, of the United States Geological Survey, to determine the effect of the amount of reinforcement and the character of the aggregate on the strength of beams and the compressive strength, modulus of elasticity, and yield point of different mixtures as to kind of aggregate and age of concrete.

The results of these tests, which were well planned and well conducted, had been completely collated and studied, and were transferred to the Bureau through the Washington office of the United States Geological Survey, December, 1910, in manuscript form for publication. Considerable editorial work was required, so that the paper did not go to the printer until July, 1911, and was not issued for distribution until April 1, 1912.

Reinforced Concrete Beams.

Series 3, 4, 5, 7, 14, 15, 16 and 23.

These series comprise tests of 537 reinforced concrete beams with the corresponding number of cylindrical compression test pieces. They were made to determine the strength of reinforced concrete beams at the ages of 2, 7, and 14 days for various aggregates and different percentages of reinforcement, the strength for various spans and proportion of concrete, the actual deformation measured on the exposed reinforcement when the beam is under load, the effect of destroying the tensile stress in the concrete for a beam under load and the effect of arching the concrete using anchoring rods. Included with these are the results of tests on plain concrete for various mixtures, ages and aggregates giving the compressive strength, modulus of elasticity and yield point.

The results of these tests which are comprehensive were transferred in the form of log sheets and largely uncomputed. They are uncollated and have not been studied. It will give you some idea of the magnitude of the work if it is stated that each beam requires from 100 to 500 calculations before any studies can be made. No work has been done by the Bureau as yet in preparing these results for publication, other than the assorting of data of the various series, but it is anticipated that some of the data can be prepared for publication during the present year.

Reinforced Concrete Beams.

Series 21, 21a.

There are 138 beams in these series with a corresponding number of cylindrical compression test pieces. They were made to compare machine and hand-mixed concrete made by the laboratory and by various contractors in the field. Gravel and limestone aggregates were used of 1:2:4 and 1:3:6 proportions tested at four and thirteen weeks.

These data are very interesting and valuable, and were transferred partly computed and collated. Some studies had been made and part of the manuscript had been written. Since the transfer the computing and most of the studies have been completed. The manuscript is in course of preparation and it is anticipated that the paper will be in press very soon.

Tests of the Absorptive and Permeable Properties of Portland Cement Mortars and Concretes and Tests of Damp-proofing and Waterproofing Compounds and Materials.

The data of this paper were collated and the manuscript mostly written prior to its transmission to the Bureau May, 1911. The studies and manuscript were completed and the paper was sent to press August 22, 1911, and was ready for distribution March 7, 1912.

The Effect of Alkali Water and Sea Water on Hydraulic Cements.

These investigations were started in 1908 to determine the action of alkali and sea waters on cements, mortars, and concretes, and were not completed at the time of transfer, July 1, 1910. The chemical investigations were completed in 1911, but the physical investigations are being continued. In the fall of 1911, the work of the Atlantic City Laboratory was moved to Charleston, S. C., to avoid the severe northern winters which affected the exposed test pieces. A paper upon this work giving the results to date is now in press and will be issued within a few weeks.

The Effect of High Pressure Steam on the Crushing Strength of Portland Cement Mortar and Concrete. Results of Tests made to Determine the Accelerating Action of Steam on the Hardening of Portland Cement Mortar and Concrete.

The uncollated data were transferred July 1, 1910. The data were collated, studied, and the paper written during the past year, being issued from the Government Printing Office in January, 1912.

Tests of Plain and Reinforced Concrete Columns.

A general series of full-size column tests were planned including investigations of unreinforced columns, and the effect of amount, disposition, form, size and position of reinforcement. This series of tests was started in 1907, and only partially completed, for it had been discontinued, prior to the transfer. The data were on log sheets and results had not been computed, collated, or studied. No subsequent work has been done by the Bureau on these tests, and it is doubtful if the results obtained will warrant publication in bulletin form.

Physical Properties of Aggregates for Cement Mixtures.

Samples of sands, stones, and gravels were collected in 1907 by laboratory representatives from various parts of the country. The deposit from which such samples were collected was examined to determine the extent and nature of the material, and laboratory tests were to be made to determine the physical properties of the material and its value as a concrete aggregate. Many materials were collected, tests were wholly or partially completed on a portion of the materials, and some of the results had been partially collated. The testing work was discontinued, sometime prior to July 1, 1910. No work has been done by the Bureau on this data other than assorting it somewhat. It is hoped that there is sufficient completed data to warrant its publication as the subject is an important one, but considerable work will be required in compilation.

Investigation of Concrete and Reinforced Concrete at Ten Technologic Institutions.

This is a compilation of tests made in 1905 at the suggestion of the Joint Committee on Concrete and Reinforced Concrete.

This report was transferred July 1, 1910, in manuscript form ready for publication. After careful consideration the Bureau did not deem it advisable to publish it; in view of the fact that the work had been done five years previous, in 1905, much of it had been published elsewhere, similar work had been done in the interim, and it was merely a compilation which is more or less incomplete at this time.

Strength of Concrete Building Blocks.

Several thousand tests were made to determine the transverse and compressive strength of hollow concrete building blocks of various types using concrete and mortar mixtures of various aggregates and consistencies. These tests were made in 1906 and 1907. Most of the results have been computed and collated, and while they are not as uniform as might be desired, they are of value in demonstrating this very quality in certain concrete mixtures, and in contributing to our present meagre knowledge of concrete building blocks. It is anticipated that these results can be prepared for publication during the present year.

A Series of Tests to Determine the Depth of Concrete Required as a Fireproofing Medium for the Protection of Reinforcement and Tests of the Fire Resisting Qualities of Typical Concrete Building Blocks and other Structural Materials.

These tests were made in 1908 and 1909, but the series was not completed. The data are uncollected and it is questionable whether the results are sufficient to warrant publication in bulletin form without further investigation. The subject is an important one and it is hoped that it may be possible to consider these data during the present year.

Investigation of Reinforced Concrete Slabs.

This comprises two series of investigations of slabs of various qualities. The work was done in 1907 and 1908, and in part at the

suggestion of a Government Department. All computations were completed and the work was partly collated. Nothing has been done with the data since the transfer, but it will be prepared for publication.

Shearing Strength of Concrete.

The tests were made in 1907. They are not very comprehensive, but give some additional data upon this subject. The results are on log sheets and have not been collated.

Tests of the Bond of Reinforcement Embedded in Concrete.

The tests were made simultaneously with the tests of reinforced concrete beams in 1907 and 1908. The data for the most part are on log sheets and uncollated.

Compressive Strength of Concretes.

Several thousand tests have been made of many different concretes in connection with the routine work of the laboratories and other investigations. There is need for information of this kind, which is of much value, and it is anticipated that these results can be compiled and published during the coming year.

Aside from routine this completes the list of data relative to cement and concrete received by the Bureau of Standards with the transfer of the work of the structural materials testing laboratories of the United States Geological Survey, July 1, 1910.

During the fiscal year of 1910 and 1911 practically no investigations of cement or concrete were conducted by the Bureau of Standards, aside from the continuation of the sea water investigations at Atlantic City, some alkali water investigations, and a short series of tests on the use of pulverized tufa in cement mortars at the Pittsburgh laboratory.

Only $50,000 had been appropriated by Congress for the testing of structural materials during the fiscal year 1910–11, whereas, $100,000 had been appropriated and expended during the preceding year. In spite of the reduction of $50,000 in appropriations, the number of routine tests made in all branches of the structural materials work was approximately double the tests of a similar nature made during the preceding year. During the

fiscal year 1911–12 the appropriation was increased to $75,000. These appropriations cover the cost of all structural materials work, such as the testing and investigation of cements, clays, limes, metals, paints, oils, varnishes, alloys, roofing materials, etc.

Routine Testing.

The number of samples of cement tested in the Bureau of Standards during the past three fiscal years is as follows:

Fiscal Year.	Number of Samples Tested.	Quantity Represented (in barrels).
1909–10	12,700	.....
1910–11	23,900	2,300,000
1911–12	19,365	2,000,000

The figures for the fiscal year 1909–10 represent the total work of the Bureau of Standards and the Structural Materials Laboratories of the United States Geological Survey prior to the concentration of the work of this character in the Bureau of Standards.

Practically all tests were made for Government departments at the Washington, Pittsburgh, and Northampton laboratories, each laboratory handling the inspection and testing of cement purchased in its particular field. The cement used in the construction of federal buildings throughout the country and miscellaneous samples received from the Government and the public were tested in Washington. Cement purchased in the vicinity of Pittsburgh by the Navy and War Departments was inspected at the mill and shipped under the supervision of the Pittsburgh laboratory. The cement for the Isthmian Canal Commission was tested at Northampton, Pa. There was a decrease in shipments to the Panama Canal during the last fiscal year which caused a falling off in the work at Northampton by 20 per cent from the preceding year. The work of both the Washington and Pittsburgh laboratories increased approximately 50 per cent over the preceding year.

Unification of Specifications for Portland Cement.

In June, 1911, the Secretary of the Department of Commerce and Labor arranged, through the secretaries of the various depart-

ments, for a conference of Government engineers for the purpose of unifying the specifications for Portland cement used by the United States Government. At this conference a committee was appointed which, after an extended series of meetings, developed a specification which was unanimously adopted by the Departmental Conference February 13, 1912. Recognizing the desirability of uniformity in specifications used by the Government and those used by the public, conferences were held with committees of the national engineering societies, and substantial agreement was reached on all but a few points.

April 30, 1912, the following Executive Order was issued by the President:

> It is hereby ordered that all Portland cement that may hereafter be purchased by any department, bureau, office, or independent establishment of the Government, or that may be used in construction work connected with any of the aforesaid branches of the Government service, shall conform in every respect to the specification for Portland cement adopted by the Departmental Conference at a meeting held at the Bureau of Standards, on February 13, 1912, and approved by the heads of the several departments (to be known as the United States Government Specification for Portland Cement): Provided, however, that such specifications may be modified from time to time by any similar Departmental Conference, with the approval of the heads of the several departments.

The desirability of uniformity in specifications is recognized not only in the advantages which accrue to the Government in obtaining a product of high quality with a reduction in cost resulting from the elimination of uncertainty of requirements, and obtain greater competition in bidding, but its effect upon the standardization of the manufactured product, which indirectly benefits the public consumer.

The Departmental Conference has been reconvened and further meetings will be held with the national engineering societies, from which, it is hoped, complete uniformity may be obtained with one specification for Portland cement as a recognized standard by the Government and the public. This conference will also act in an advisory capacity in connection with the Bureau's investigations of the physical properties of cement and concrete.

The following investigations are now being made by the Bureau:

1. Composition of Portland Cement.

A series of investigations are being conducted at the Pittsburgh laboratory on fundamental problems concerning the composition, and manufacture of cements. From this investigation it is anticipated that the effect of variation of the elements in cement, and the effect of method of manufacture may be determined.

2. Investigation of the Effect of Sea Water and Alkali on Portland Cement.

The investigation of the action of sea water on cements, mortars and concretes is being continued at Charleston, S. C. A technical paper is now in press giving the preliminary results of the laboratory and field investigations.

The laboratory investigation of the effect of alkali salts on cements has been completed, and the results are included in the above publications. It will be necessary to make a field investigation in continuation of the laboratory work, and this is highly desirable at the present time, as a large percentage of the farm lands in the West which are under irrigation, are subject to alkali concentration unless properly drained, and the cheapest and most available construction material is concrete if it can be safely used. Public and Government engineers throughout the western part of the United States where alkalies are present are for the most part very skeptical of the suitability of concrete, for there have been many failures of concrete structures. It is anticipated that with the cooperation of other Government departments and the state experiment stations, a comprehensive investigation may be inaugurated during the present year.

3. Investigations of Fine Grindings of Cements and Sand Cements.

It has been claimed that a large percentage of Portland cement (the coarser particles) have very little or no cementing value. At the present time there is no method of analyzing a cement by which its cementing value may be definitely determined by separating the more or less so-called inert material from the active material. The Bureau is endeavoring to determine the cementing value of the various sized particles in cement. This necessitates

the development of air-separating apparatus which may be substituted for the sieve in mechanical analysis. In this connection the effect of grinding sand or other puzzolana material with cements is also being investigated. This investigation is of prime importance to such divisions of the Government as the Reclamation Service, where the freight charge in transporting cement may exceed the value of the cement, and a considerable saving could be made by separating and shipping only active cementing material, if this be possible. The product known as sand cement is now being used by the Government, but without very definite knowledge of its cementing value.

4. The Value of Steam Treating Cement or Concrete.

An investigation is being made to determine the effect of heat moisture and pressure independently or combined upon the hardening of cement and concrete. The results of this investigation may be of prime importance to the concrete products manufacturer, who uses steam curing for accelerating the hardening of his products. It has been proposed that the specification for the purchase of cement be amended to include a test in which the cement is exposed to high pressure steam. In this connection an investigation is being made to determine the value of such a test as a means of indicating the structural quality of cement.

5. The Permeability of Structural Materials.

An investigation of the permeability to water of structural materials is being made, which is now, in part, a continuation of the work reported in Technologic Paper No. 3. This information is much needed by the Government as well as the public, in designing various structures where impermeability or protection against dampness are a necessity. The work involves both laboratory and field tests.

6. Volumetric Changes in Concrete.

The investigation of the volumetric changes which take place in concrete structures, owing to temperature variation and that which takes place during the hardening of concrete is being made as a means of determining the necessity of expansion or contraction

joints or the causes of cracking in concrete structures. In this connection laboratory experiments and field investigations are being made by placing reference marks on structures which can be measured from time to time, and volumetric changes determined.

During the past fiscal year four publications on the subject of cement have been issued:

Technologic Paper No. 2. Results of Tests on 333 Reinforced Concrete Beams.

Technologic Paper No. 3. Tests of the Absorptive and Permeable Properties of Portland Cement Mortars and Concretes, together with Tests of Damp-proofing and Waterproofing Compounds and Materials.

Technologic Paper No. 5. The Effect of High Pressure Steam on the Crushing Strength of Portland Cement Mortars and Concretes.

Circular No. 33. United States Government Specification for Portland Cement.

Three publications are now in press:

Technologic Paper No. 12. The Effect of Sea Water and Alkali Water on Cements, Mortars, and Concretes.

Circular No. 39. Specifications for the Measurements of Standard Sieves.

Technologic Paper No. 18. The Effect of Electric Current on Concrete.

While the major portion of the Bureau's work in the cement field in the past has been the routine testing of cement, and while the routine testing is of much value to the Bureau in keeping in touch with the problems of the industry, every endeavor will be made to expand the research investigations as rapidly as possible. The preparation for publication of the data transferred from the United States Geological Survey will be expedited.

The possibility and advantages for cooperation in the Bureau of Standards are most excellent, and there is no division of the work of the Bureau which cannot be of service to the cement industry. The Electrical Division has been investigating the

effects of stray electric currents upon concrete, and the metal reinforcement of concrete structures, and its effect in general upon all steel structures, water piping, gas piping, etc. The Heat Division is determining the general heat constants. The thermal conductivity of cements, mortars, and concretes, and building materials in general is of prime importance. With the improvement and standardization of chemical methods and the issuance of standard samples, the Division of Chemistry is of service to the cement industry. The Division of Weights and Measures has demonstrated its possibility in the standardization of sieves, and the development of a new sieve specification. While the work of the Optical Division has no direct bearing upon the cement industry, it is not unlikely that it will be found of service in developing interferometers or other measuring apparatus for detecting slight deflections or volumetric changes in concrete structures.

As cooperation is the keynote of the times in modern industry, it is believed that the coordinating of the investigations of structural materials with the work of the Bureau of Standards is highly advantageous.

SAND AND GRAVEL WASHING PLANTS.

By Raymond W. Dull.*

There has been a very marked change, during the last few years, in preparing materials for concrete purposes. We seldom see the old obsolete method of sending a team to a river bank, or pit for gravel aggregate, regardless of size, proportion or cleanliness. We now see a modern factory for handling, sizing and washing the gravel that decreases the cost of handling, prepares it for different kinds of uses, and, best of all, makes it suitable for the purpose.

Unclean material, with poor workmanship, has been responsible for nearly all failures. A contractor who is not particular about his materials is seldom particular about his workmanship. It is useless to buy high-grade cement and mix it with gravel having even a thin film of dirt coating the pebbles, for the cement cannot adhere to the pebbles.

The successful method used in the modern gravel washing plant is to agitate the material until the impurities are in suspension in the water, drawing the water off continually, and then rinse the material as it leaves the screens. It is important to leave the water with the material until the last, in order to get sufficient agitation, and that is why separate screens are used for each size.

Screens.

The writer, after considerable study, developed the hook-bolt type conical washing screen (Fig. 1), which, on account of its simplicity, low power, efficiency and durability, has served the purpose exceedingly well.

These screens are set in a series (Fig. 2) with spouts carrying the sized material to succeeding screens below, while the rejections from the screens discharge from the mouth of the bell into the bins below. The material gets its rinsing from fresh water pipes at the mouth of the screens.

The upkeep of this screen is low because the bearings and

* President, Raymond W. Dull and Company, Aurora, Ill.

drives are away from the grit and water. The large hook-bolts which support the screen cone do away with all fastenings to the casting, and the screen cones can be changed in less than an hour's time.

Appreciating the advantage gained if all chain drives could be eliminated, the inclined conical screen was conceived, a still more radical departure in screen construction. All the screens in this case are mounted on one shaft, and a pair of gears drives everything (Fig. 3). This type of screen differs in its operation from the other screen, for the material is delivered into the large end and travels toward the small end, while with the hook-bolt type the material is delivered into the small end and travels toward the large end.

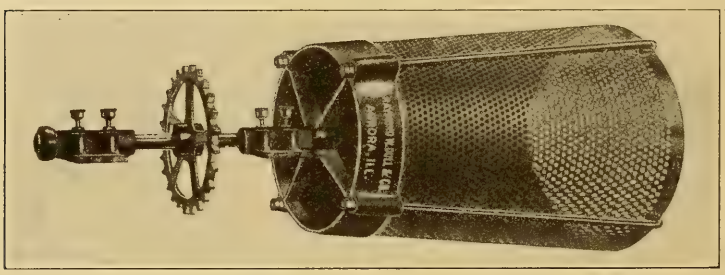


FIG. 1.—CONICAL SCREEN OF HOOK-BOLT TYPE.

The inclined conical screen has the advantage because the large end does the principal work. The large end has more perforations, more wearing surface, and is in fact equivalent to a larger screen. It therefore has greater efficiency, requires less power and has small upkeep. The screens have longitudinal joints, and can be dismantled from the shaft without disturbing it. Walkways are also provided on both sides of the screens, to make them accessible.

Water Supply.

The largest quantity of water is introduced when the material is first starting into the screens, and additional fresh water is added at each screen. The water supply is a very important detail of the plant, and sometimes success or failure depends entirely on this feature. After the material passes through all

the screens, the remaining materials are the soil water and the sand.

SETTLING TANK.

A device extensively used for separating the sand from the soil water is the wooden tilting tank of Charles Stebbins, of Chicago. This tank rocks on two bearings on the sides of the tank. It is divided into two triangular compartments, one for the water and the accumulation of sand, and the other compart-

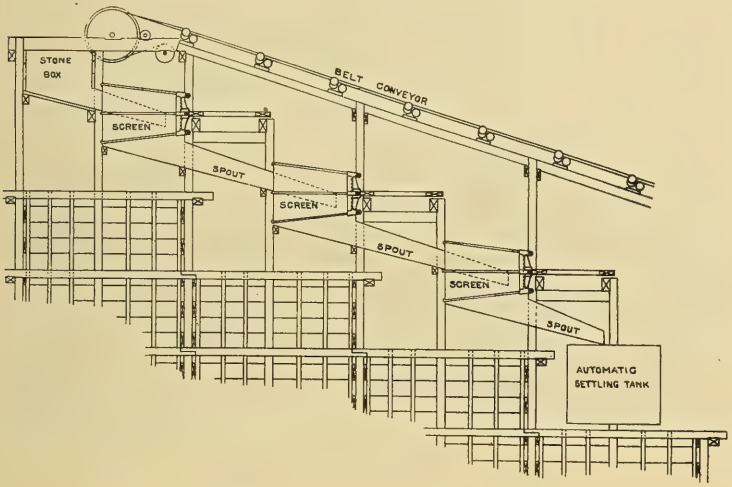


FIG. 2.—ARRANGEMENT OF CONICAL SCREENS.

ment for counterweights, acting counter to the weight of the water and sand.

The valves of the tank consist of pipes, fastened to the tank, with their ends bearing against a fixed timber. When the sand accumulates enough weight, the tank tilts, drawing the pipes from the timber stop, and allowing the sand to escape into the bins below. When a certain amount of sand escapes, the tank tilts back, closing the valves.

A modified form of tilting tank made of steel, which is somewhat more sensitive and designed to permit adjustment of the equilibrium, is shown in Fig. 4. It consists of a triangular shaped tank, rocking on two knife-edged brackets on the sides, and

counterbalanced by a lever and weight. The lever is designed to increase the moment arm of the weight, which retards the tendency to tilt open too far, and does not permit flushing of the soil water into the sand bin below. The valve castings, which have "V" edges, bear against steel plates with rubber faces, which are pivotally connected to timber. The operation of these tanks is very gratifying and results are very satisfactory.

WASHING THE AGGREGATE.

The reason it is so difficult to wash sand and gravel in the aggregate is simply this: When getting rid of the water, it must

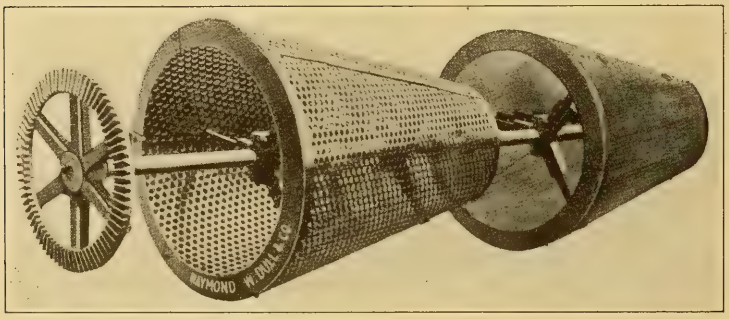


FIG. 3.—INCLINED CONICAL SCREEN.

not be allowed to percolate through the material to drain off. To do this makes a regular filter bed operation, and all the impurities in the water are held in the material.

Of course a constant stream of water can be added, until practically all the impurities are taken away, and then the clear water can be drawn off at the bottom. This means new material must not be added, and it reduces the operation to a batch or intermittent method. The quantity of water necessary also is excessive. There are devices on the market for mechanically lifting the material out of the water, which produce material washed very well, but the power and upkeep of this class of machinery is very great. It is also impossible to get the water from the aggregate mixture by using a tilting tank, because the gravel will not allow the device to work properly.

It is better to wash the material through sizing screens and then mix together as the material comes from the screens. A plant was designed to segregate the material into separate bins, or combine them together in one bin for use in reinforced concrete work. This plant assembles all the material, when desired, into the central bin. The material from the first screen is not included, on account of its large size. The proportions are

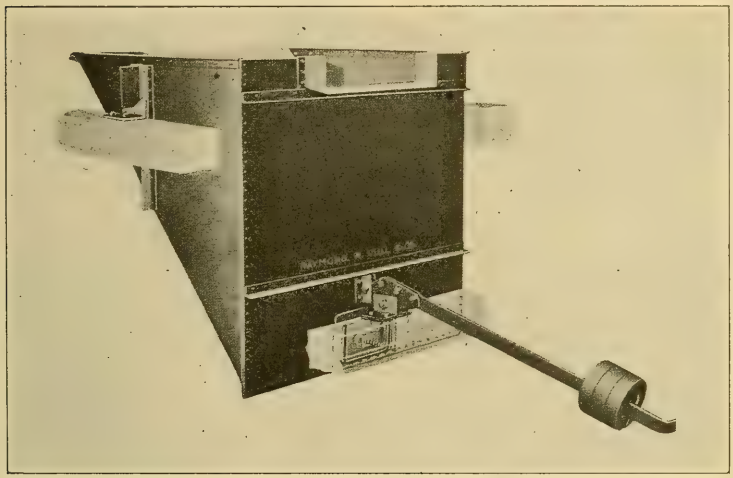


FIG. 4.—TILTING SAND SEPARATOR.

regulated by the slide gates, which cut off so much of the flow, and shunts the remainder into the other bins.

MIXING.

Great care must be taken, however, in relying on a mixture accumulated in bins, for the mixture of material may be perfect in the bins, but separated to a distressing extent, when it is drawn from the bin. The coarse material will flow more readily and go to the far side of the car or wagon, and the fine material, which is more sluggish, will drop into the car on the near side. The result is that one side of the car has coarse material and the other side fine material.

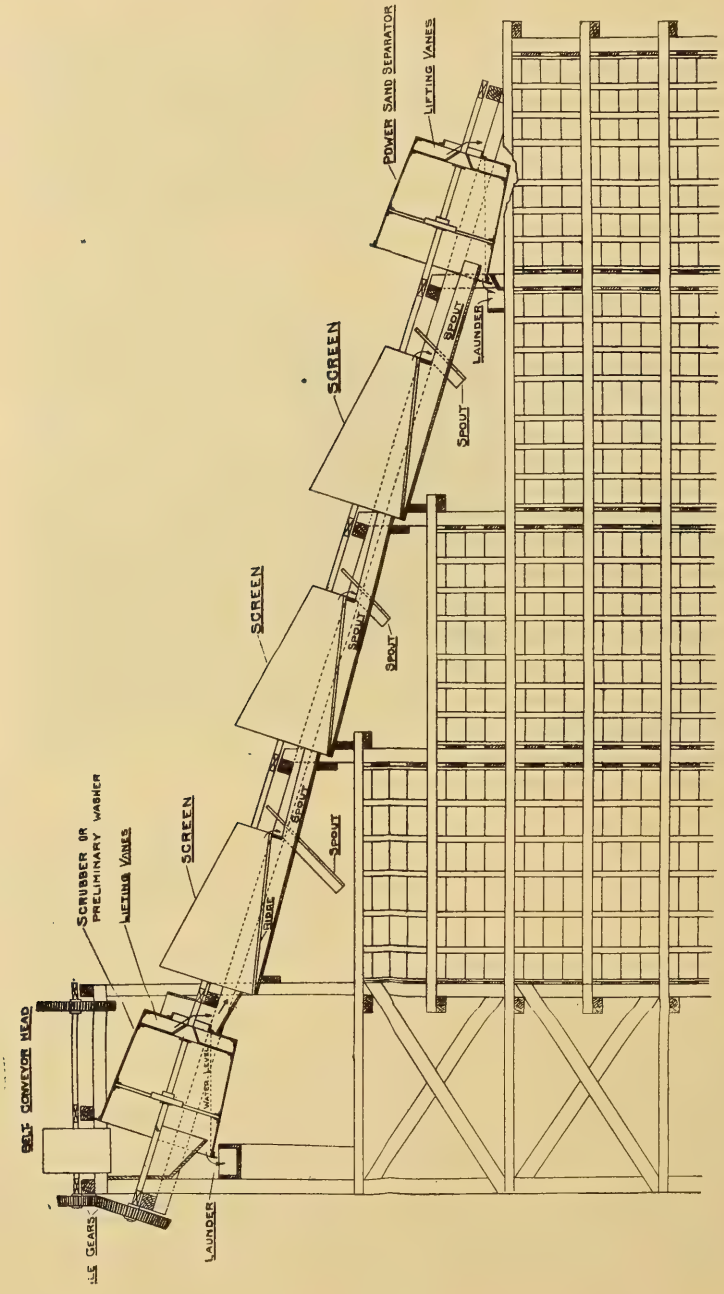


FIG. 5.—PLANT WITH PRELIMINARY SCRUBBER.

SCRUBBER.

So far the washing problems have been attacked from a mechanical standpoint, but the aid of chemical analysis is appreciated. There are some materials which have impurities very hard to eliminate. Some clays will not succumb to ordinary processes. This led to the introduction of a preliminary scrubber for materials unusually hard to wash, Fig. 5. It consists of a frusto-conical drum so shaped as to retain considerable material and water, which

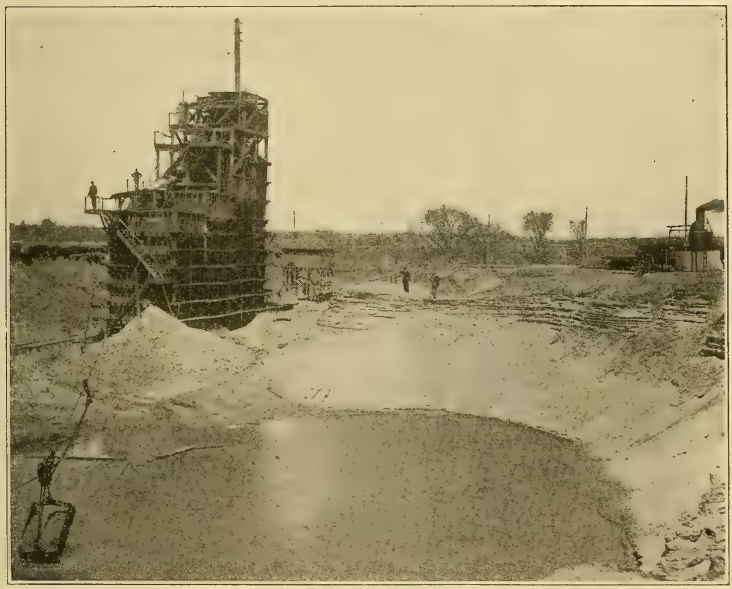


FIG. 6.—PLANT WITH DRAG-LINE EXCAVATOR.

tumbles about during rotation. This agitation breaks up the clay and the worst part of the impurities are carried off before the material reaches the screens.

This same device used as a mechanical sand separator is also shown. The separator is hardly as efficient as the tilting tank and also has the disadvantage of requiring power, but it has the advantage that the sand bin can be made 5 or 6 ft. higher.

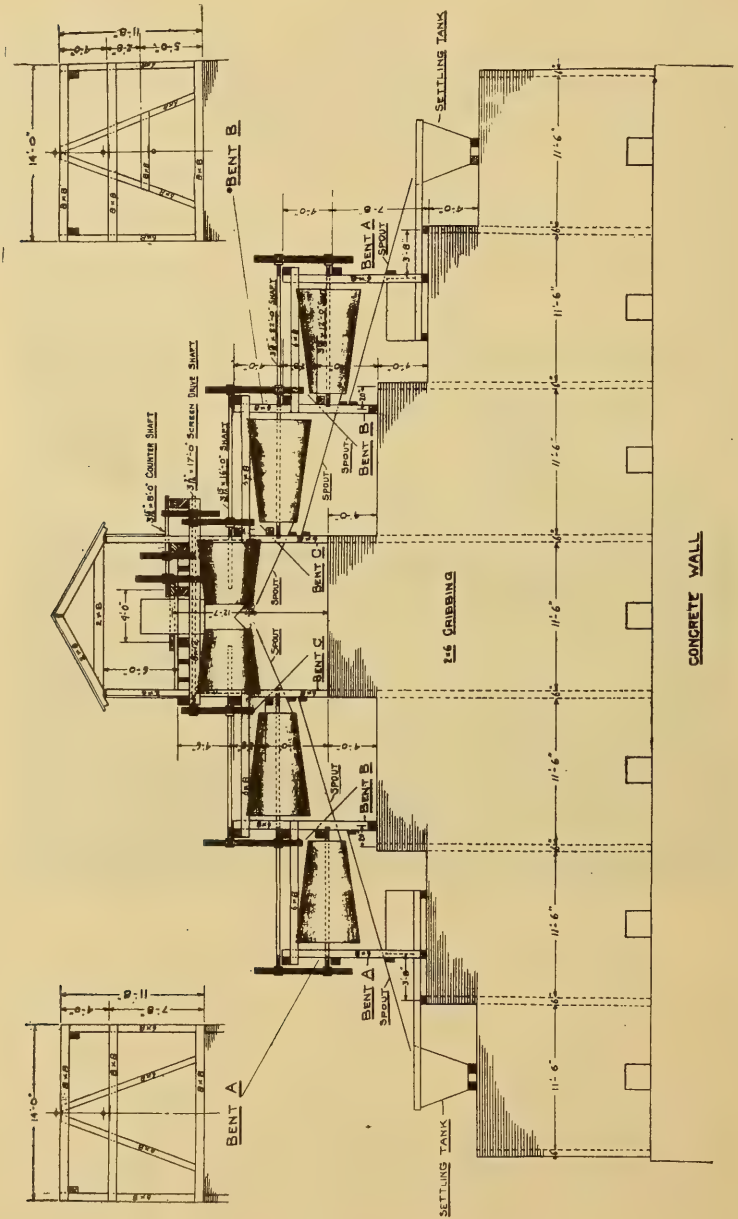


FIG. 7.—BINS OF CRIB CONSTRUCTION.

Methods of Handling.

The belt conveyor is almost universally used on the larger plants for carrying the material to the top of the bins (Fig. 2). It is efficient, and if properly installed is very satisfactory for this purpose. It has been found, however, that the standard troughing idlers wear out very rapidly in handling this kind of material, and a pressed steel pulley was devised, with which to equip the idlers. Thus equipped they are able to carry large boulders, and being split they are readily changed.

Belt conveyors are also used to convey material to the plant from the excavator. Drag-line excavators are also coming into use for plants of moderate capacity, Fig. 6. They have the advantage that one man at the hoisting engine can elevate and deliver the material to the plant. The plant shown is equipped with a machine which returns by gravity very rapidly.

If the material is below water, sand pumps are extensively used. This method gives the material a preliminary washing, but used with flat screens, which is the common practice, does not size accurately, and there is usually considerable sand in the gravel. The sand is not always clean and in some cases the material is pumped twice to accomplish the washing. The upkeep of a pumping plant is very high, as is also the power required. Some of the companies using pumps are installing revolving screens, with much better results than with flat screens.

Bin Construction.

In constructing bins, the skeleton construction is preferable to the cribbed construction. The skeleton design, Fig. 6, can be repaired much easier and requires less material to make. Solid plank walls, built up by spiking the planks flat-wise, one upon the other, is what is known as crib construction. The reason that this construction is not desirable is because so much water in the bins remains in the joints between the planks, and rots them very rapidly. If this construction is used, all joints should be well tarred (Fig. 7).

A few washing plants have reinforced concrete bins. Concrete makes excellent bins, and of course they are more permanent than those made of wood. Circular steel tanks also make excel-

lent bins. The bin construction is usually a matter of preference with the purchaser.

LOADING DEVICES.

The sand and gravel is delivered into cars usually by steel counter-weighted spouts, although some bins are constructed over the track and the sand and gravel is dropped directly down into

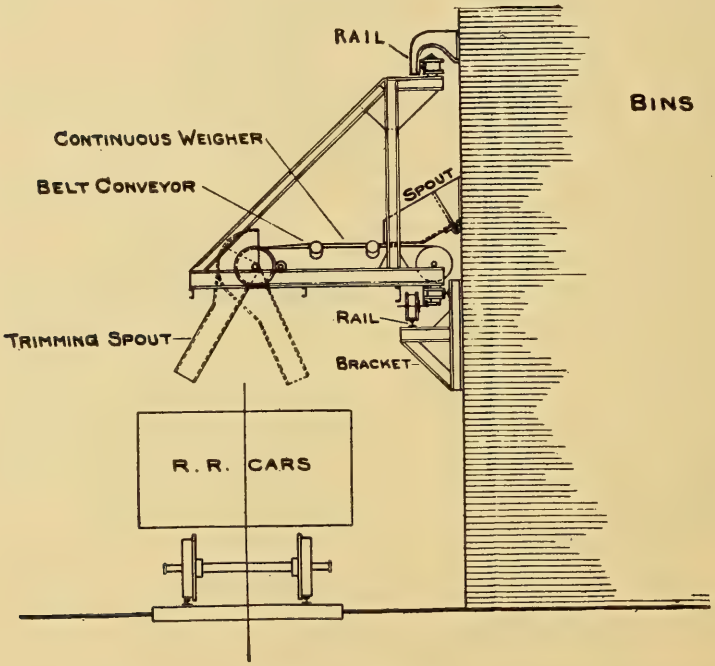


FIG. 8.—LOADING DEVICE WITH CONTINUOUS AUTOMATIC WEIGHER.

the cars. In shipping material in this manner there is always danger of overloading, and one is compelled to rely on railroad weights when shipping.

In order to weigh the material as it is loaded, there was lately designed a loading device, equipped with an automatic weigher. It consists of a short belt conveyor, mounted on a frame, very similar to a wall crane, Fig. 8, which travels on the face of the bins, and will stop in front of any gate opening. A

section of the belt passes over the weighing apparatus, and the quantity which passes over the machine is weighed. The dial which records the weight reads like a meter, and the weight of material in each car as well as the total amount loaded for the day is recorded. In addition to this advantage, the machine has a swinging loading spout, which spreads the material in the cars and dispenses with considerable hand shoveling.

It is gratifying to observe that practically all the new plants constructed are washing plants, even in places where the material is remarkably clean.

The necessity and importance for standardizing and grading sand and gravel cannot be overestimated and is made more apparent by reason of the fact that committees of this Association as well as the Gravel Producers' Association are endeavoring to prepare proper standards.

TOPICAL DISCUSSION.

The President. PRESIDENT RICHARD L. HUMPHREY.—The purpose of this discussion is the interchange of experiences in the solution of problems arising in actual work, by asking and answering questions.

The construction of concrete highways is one of the most important matters this Association has to consider, especially as to the methods to be employed and the cost of construction and maintenance.

As these roads are built and experience is thus gained, improved methods of construction will be introduced, thereby increasing the durability and decreasing the cost of construction and maintenance.

The history of the use of concrete sidewalks will be repeated in the use of concrete roads until, as with sidewalks, its use for road building will be universal.

Considering the number of miles of roadways in the United States, the very few miles that are improved and the acknowledged excellence of concrete roads, you can readily see what a great future there is for these roads.

Dusting is an important matter to every one interested in the construction of floors, sidewalks and roadways. This dusting is not so noticeable in sidewalks and roadways as it is in the floors of buildings. In some manufacturing plants this fine dust is detrimental not only to the machinery but also to the process used.

A full, frank discussion of these matters cannot be other than helpful.

Mr. Talbot. MR. K. H. TALBOT.—Mr. President, last year in the discussion of concrete floors the suggestion was made that the surface be ground with carborundum stones in order to do away with dusting. It would be of interest to know if any one has tried this during the year, and whether they have any suggestions to offer. I inspected one rather exceptional piece of floor construction during the past year. The Borland Interests in Chicago, after deciding upon the use of concrete floors in their industrial buildings made a very thorough investigation of the subject and

inspected floors in the East and West, and after conferences with experienced engineers, architects and contractors, it was decided to accept the specifications adopted by the National Association of Cement Users in 1912, which require a top course mixed one (1) part of Portland cement to two (2) parts of aggregate for wearing surface; the aggregate for wearing surface consisting of three (3) parts passing a $\frac{1}{2}$-in. and retained on a $\frac{1}{4}$-in. screen, and two (2) parts passing $\frac{1}{4}$-in. screen and graded down to dust. Granite aggregate was used throughout for base and wearing surface. The mortar was mixed quite dry and was thoroughly worked with a wood float before finishing. In no case did more than three hours elapse from the time of mixing until the finishers left the floor. As soon as the floor had hardened sufficiently to allow sprinkling without pitting the surface, it was wet and covered with two inches of sawdust which was kept wet for two weeks. That the flinty hardness of the floor was due in no small measure to curing under the favorable conditions of being covered and kept moist, is evident from the softness of one spot where the sawdust was accidently removed and not replaced, allowing the floor to dry out rapidly. Mr. Talbot.

It has been my experience that sidewalks are not always as satisfactory as they should be, especially in small communities where there is no expert supervision of the contractor's work. The tendency of all contractors on sidewalk work is to spend most of their time and care on the finish, and it often happens that the base is not sufficiently well mixed, nor is there a sufficient amount of cement to keep it from being porous which permits frost action to break up the walk. Again, the base is allowed to harden too much before the top is placed. It seems to me that this Association might well start a movement for one-course sidewalks. Experience goes to show that one-course roads are entirely satisfactory and that it is possible to give them any kind of a finish necessary. I believe that the same method can be employed for one-course sidewalks that is being employed for one-course roads, and that we will thus get a uniform walk which will not be affected by frost and will not have a tendency to crack at the weak plane between the top and the base.

Mr. Allen Brett.—When we see a four-inch slab poured on a floor, we wonder why a one-course slab for sidewalk would not Mr. Brett.

Mr. Brett. work. Personally I have always believed in a one-course sidewalk; in the summer of 1911, the City Engineer of Salt Lake City specified considerable work which was carried out under a one-course specification and the work was as well done as could be expected, using a wet concrete; as, however, it was not as cheap as two-course work, the specifications of 1912 have gone back to two-course work. As far as I know Salt Lake City is the only city that has tried one-course work, and it was not satisfactory. It seems to me that one-course work should cost less than two-course work. I have written the City Engineer of Salt Lake City, asking why they should be able to lay two-course work for less than one-course work but so far I have not heard from him.

Mr. Whited. Mr. Willis Whited.—With regard to one-course work, I think a little distinction is necessary. In city work, in a residence district a sidewalk wears so little that one-course work seems to me rather extravagant; in the business districts where the traffic is heavy and the sidewalks really wear quite appreciably, in some cases quite deeply, then one-course work would be desirable. My idea is that the one-course sidewalk would stand the most wear if it is made of the right material, but that the two-course sidewalk would cost less to construct.

Mr. Schouler. Mr. W. W. Schouler.—I would like to know the difference between the cost of one- and two-course work. I have always claimed that you could save enough in labor to offset the cost of the cement required. In the one-course work a 1 : 2 : 4 mix would offset the 1 : 2 wearing surface and the 1 : 3 : 5 base, and the labor of the average two-course work is three cents a superficial foot, while one-course work can, I believe, be done for two cents a foot. I do not see why a one-course walk will not stand wear as has that in front of the Hotel Astor in New York for the last ten years. The coarser aggregate in the surface takes the wear. It may look a little rough, but for good, hard wear and tear one-course will last longer than two-course work.

Mr. Talbot. Mr. Talbot.—I find that for 100 sq. ft. of 4-in. one-course sidewalk mixed one (1) part of cement, two (2) parts of sand, and four (4) parts of stone or gravel, there will be required 1.86 barrels of cement, 0.55 cu. yd. sand and 1.11 cu. yd. of stone; while for 100 sq. ft. of two-course sidewalk 4 in. thick made up of

$3\frac{1}{2}$-in. base, mixed one (1) part cement, $2\frac{1}{2}$ parts sand and five (5) parts gravel or stone, and $\frac{1}{2}$-in. top, mixed one (1) part cement to two (2) parts of sand, there will be required 1.85 barrels of cement, 0.64 cu. yd. of sand, and 0.99 cu. yd. of gravel or stone. It will be noticed that there is a difference of only 0.01 barrel between the quantity of cement required for a one-course and a two-course walk, while the amount of aggregate is practically the same. In one-course work the reduction in cost of placing, due to the fact that it is only necessary to have one mixer, thereby doing away with the necessity of constant supervision to ensure the right mixture being used, will more than compensate for any difference in the cost of extra tamping or floating in two-course work. Mr. Talbot.

President Humphrey.—We have formed a habit of constructing sidewalks in two courses; when we acquire that habit of one-course work we will find the result equally (and in my opinion more) satisfactory. The President.

Mr. Schouler.—I believe that a still cheaper walk can be made by using two-inch stone tamped into place and grouted with 1 : 2 mixture, thus eliminating half of the labor. The stone should be wet with water to insure the grout going to the bottom of the pavement. Mr. Schouler.

Mr. Talbot.—Referring to the question of grouted sidewalk, we are all familiar with the method employed by the Hassam Paving Company in the construction of their pavements. It seems to me from what I have seen of this construction that there is a good deal of what might be called personal equation in the work. The results must depend largely upon the superintendent and upon his laborers. The stone must be of the right size, and mortar must be mixed to just the right consistency. It has been the experience of this company that they cannot mix cement grout satisfactorily by hand because of the necessity of having it very wet, and this has resulted in the design of a special grout mixer. If this grout is too dry it will be impossible to get satisfactory penetration. It is not as though the contractors who do sidewalk work were engineers who could make such changes as will be necessary to get results. We must write specifications for general use, and I do not believe that this Association ought to go on record as favoring a grouted sidewalk. Mr. Talbot.

The President. President Humphrey.—There is a tendency at the present time to use less water for mixing concrete than is the current practice. When this Association was organized (1905) it was the practice to mix quite dry using very little water; in the first conventions one risked his life in advocating wet concrete. Since that period the pendulum has swung until now too much water is generally used. The time is not far distant when longer mixing and less water will be the practice. While grouting may be used under certain conditions it is not desirable. As little water should be used as will secure, with thorough mixing, a concrete of a viscous consistency. Such a concrete will make highly satisfactory one-course work. To place a brittle wearing surface on pavements is a mistake. What is required is a concrete slab of a high uniform density and strength and having a plane rough surface. Such one-course work will be the cheapest and most satisfactory for sidewalks and roads.

Mr. Schouler. Mr. Schouler.—I have tried large and small mixers and am using now two or three mixers of about a wheelbarrow capacity, that are peculiarly adapted to curb and gutter work, where you cannot use a larger mixer. The mixture can be made as accurately as you desire, it mixes well, is inexpensive, and the men do not have to work so hard, but rather enjoy it.

Mr. Whited. Mr. Whited.—The problem of sidewalk construction in Pittsburgh is a little different from what it is in a good many places. Pittsburgh has some very heavy grades, and in some districts the traffic is heavy and considerable wear must be provided for. The practice here is to put in a good foundation, provide thorough drainage, then a base of 1 : 3 : 6 concrete, and a 1 : 2 surfacing of Ligonier screenings. Ligonier stone is a lime cemented very hard sandstone almost as hard as granite. I suppose granite would be a little better but it costs a great deal more than Ligonier. Most cities use a smooth finish and run a marking roller over it; but here they finish with a wood or a cork float which gives a sandy finish that is not slippery, and is still smooth enough to make it easy to shovel snow or ice from it; it does not wear smooth for a good many years. It is sometimes difficult to get the contractors to cut through the joints so that they will not buckle in hot weather; the top coat peels off sometimes, because, probably, the contractor is a little slow in placing the top coat which does not adhere properly to the under coat

Mr. Schouler.—Has anyone had experience with the specifications eliminating the cinder foundation, that were adopted by this Association about five years ago. I have for five years been laying sidewalks that I have guaranteed under those specifications and have yet to repair one; while I frequently had to repair those that were laid on the cinder base. I believe now, as I contended five years ago, regardless of weather conditions or of soil, that is the proper way to lay a sidewalk. If you use iron forms and separate each slab, making the latter as many inches thick as it is feet long or broad, you have a pavement that will not crack; frost will not lift an individual slab more than a quarter inch, which will go back again in the spring. Mr. Schouler.

Mr. Talbot.—Mr. President, I would like some discusssion of aggregates, with particular reference to the necessity of using either a hard granite or trap-rock screenings, or even limestone screenings in place of sand. Mr. Talbot.

President Humphrey.—The wearability of sidewalks is entirely a question of workmanship and of the hardness of the aggregate used. The pitting that is sometimes noticeable results from the use of aggregate containing soft particles. The ideal pavement is one in which there is hard aggregate to take the wear embedded in sufficient Portland cement to properly bind it together and hold it in place. The President.

Mr. Brett.—Has any member of this Association investigated the feasibility of securing the effect of a terrazzo finish by grinding the surface of the concrete with carborundum, so as to expose the aggregate? At the New York Cement Show, a year ago, there was a rotary machine that produced a fine, smooth, surface on stone or gravel concrete. Mr. Brett.

Mr. A. J. Maynard.—I have had quite a little experience rubbing the surface of concrete. I have had many samples made by the carborundum manufacturers with which to rub the surface on terrazzo floors, and I have not found any that would answer the purpose. We use, generally, Nova Scotia sandstone; sometimes we use loose sand when the top of floor gets too hard. The tendency of the men is to use a pretty wet concrete for the top surface; it is done with unskilled labor when there is a rush to get the work finished; if troweled very wet there is a tendency to leave trowel marks. When the floor has set up hard, say in Mr. Maynard.

Mr. Maynard. three or four weeks, we use sandstone to rub the surface smooth, and to remove all trowel marks. The finish is used for interiors, but it can be used for exteriors. Floors finished in this way never dust. Our labor cost does not enter at all, as it is a state institution. We generally use a half inch of 1 : 2 : 5 wearing surface on reinforced floors; in basement floors we use a 1 : 4 : 8 foundation, and a half inch of 1 : 2 : 5 wearing surface. The material used is neither a trap rock nor a granite but is what we call a bastard stone; it is sometimes hard and sometimes soft and frequently contains pieces of slate. The material all passes a $1\frac{1}{4}$-in. screen and is retained on a $\frac{1}{4}$-in. screen. We do not use any finer than $\frac{1}{4}$-in.

The President. President Humphrey.—Mr. Maynard has brought out a point worthy of consideration. There is a tendency to use too much cement. The durability of a floor or sidewalk will depend largely on the care and thoroughness with which the concrete is mixed and placed. If the work is hurriedly and improperly performed a larger amount of cement may be necessary to, in a measure, off-set this. When concrete is properly and thoroughly mixed and placed it is surprising what a relatively small quantity of cement is required to secure a given strength. I recall some experiments with the cement gun which confirm this statement and which showed that a 1 : 4 mixture applied by this method was very much stronger than a 1 : 2 mixture applied by hand. The explanation is, that through the more complete elimination of air and water voids, the ingredients of the mixture have been brought into closer contact, thereby increasing its density and consequently its strength. The function of the cement in mortar or concrete is to bind the aggregate together. It does not of itself possess the strength of a piece of quartz, granite or trap rock. Neat cement has a crushing strength of about 10,000 lb. per sq. in., while the strength of granite is more than twice that amount. It is evident therefore that adding cement will not increase the strength of granite concrete except in so far as it decreases the voids and increases its density. The aggregates should be so graded and proportioned as to reduce to a minimum the voids and the cement required.

Simply stating the proportions of cement and aggregates does not indicate whether the mixture is a proper one or not.

The President. It is a question of density; a lean mixture well graded and mixed will be stronger than one poorly graded and mixed that is richer in cement. The amount of mixing which eliminates or reduces the air and water voids is an essential feature in securing density. A lean concrete that is compacted by vibrating will be very much more dense and stronger than one containing much more cement that is mixed by hand methods.

PROPOSED STANDARD SPECIFICATIONS FOR THE DESIGN AND CONSTRUCTION OF STONE, CONCRETE, AND REINFORCED CONCRETE HIGHWAY BRIDGES AND CULVERTS.

Steel.

All reinforcing steel shall be of structural steel grade as called for by the Standard Specifications of the American Society for Testing Materials.

Cement.

All cement shall conform to the Standard Specifications of the American Society for Testing Materials.

Aggregate.

All sand, gravel and broken stone shall conform to the specifications of this Association.

Unit Stresses.

The unit stresses used in designing shall conform to the recommended practice for reinforced concrete of this Association.

The unit compressive stress in arches, including temperature stresses, shall not be greater than 37.5 per cent of the compressive strength of the concrete at 28 days, or 750 lb. per sq. in. on 2,000 lb. concrete.

Loading.

Dead Load. The following unit weights shall be assumed in calculating dead loads:

Material.	Weight per Cu. Ft. (lb.).
Earth filling	130
Plain concrete	150
Reinforced concrete	155
Steel	490
Cast iron	450
Vitrified brick	125
Common brick	115
Granite masonry	175
Sandstone and limestone masonry	160
Macadam and Telford	110
Pine, fir, etc	42
Oak and yellow pine	48
Creosoted wood	60

The weight imposed by earth filling shall be assumed to be the weight of the filling vertically above the portion of the structure in question, unless the depth of the said filling exceeds one-half the span, in which case the assumed depth of the filling shall be equal to one-half the span In the case of spandrel-filled arches, if two planes inclined at an angle of 30 deg. with the vertical, one tangent to the extrados of the arch on each side, shall intersect below the surface of the road, only that part of the filling below said planes shall be considered, otherwise the whole of the filling vertically above the arch shall be assumed to be resting upon it. In determining the passive resistance of the earth filling over arches, due regard shall be paid to the friction between the particles of the same.

Live Load.

Class A Bridges, for ordinary country roads.

The load for the roadway floor and its connections and for spans not exceeding 60 ft. shall be assumed not less than 100 lb. per sq. ft., or a 10-ton traction engine whose axles are 8 ft. centers and wheels 5 ft. gage, 6 tons on rear axle and 4 tons on front axle, occupying 10 x 20 ft. The main trusses of spans exceeding 60 ft. shall be designed to carry the following loads:

Length of Span in Feet.	Load per Sq. Ft. (lb.).
60 to 80	90
80 to 100	80
100 to 150	70
150 and over	60

Class B Bridges, for main thoroughfares and improved roads.

The load for the roadway floor and its connections and for spans not exceeding 60 ft. shall be assumed not less than 125 lb. per sq. ft., or a 16-ton traction engine whose axles are 10 ft. centers and wheels 5 ft. gage, 10 tons on rear axle and 6 tons on front axle, occupying 10 x 24 ft. For spans longer than 60 ft. the trusses shall be designed to carry the following loads:

Length of Span in Feet.	Load per Sq. Ft. (lb.).
60 to 75	110
75 to 100	95
100 to 150	80
150 to 200	70
200 and over	60

The load on sidewalks shall be taken as 90 lb. per sq. ft., but not less than 400 lb. per lin. ft. for each sidewalk.

Special Bridges.

City bridges and bridges carrying traffic connected with mines, quarries, lumber regions, mills, manufactories, etc., shall be designed to carry any specially heavy loads which may reasonably be expected to pass over them.

Bridges carrying interurban electric cars shall be designed to carry cars weighing at least 50 tons, with trailers weighing at least 30 tons, the axle spacing to be as the cars are actually built.

Stringer Loading.

Stringers shall be designed to carry the full load of the wheel or wheels resting directly upon them, unless the centers of the adjacent stringers are within 20 per cent of their length of the loaded one, in which case each adjacent stringer shall be assumed to carry a portion of the load determined by the following formula:

$$l = \frac{20 - p}{4}$$

in which

l = load carried by each adjacent stringer.

$$p = \frac{100 \times \text{the c. to c. spacing of the stringers}}{\text{length of each stringer.}}$$

provided the reinforcement is continuous over the stringers.

A concentrated load resting on a slab supported on two opposite sides only shall be assumed to be distributed over a width of the slab equal to one-fourth the span of the slab in addition to its own width and to any distribution due to the filling and paving on top of the slab. The secondary reinforcement of the slab shall have an area equal to at least one-fourth of the area of the principal reinforcement per unit of width.

The axle loads in the case of street cars shall be assumed to be distributed over the whole area enclosed between three successive ties.

Loading on Arches.

Concentrated loads resting on spandrel-filled arches shall be assumed to be vertical and distributed through the filling according to the rule given below for flat slabs, except that the distribution due to the thickness of the arch sheeting shall be considered equal to $1\frac{1}{2}$ times the thickness of the sheeting.

Concentrated loads resting on earth filling or paving on top of slabs or arches shall be considered to be distributed beyond the

limits of the load in all directions a distance equal to seven-tenths of the depth of the filling.

Stresses in Arches.

Arches shall be considered to be hingeless and calculated according to the elastic theory unless hinges are provided.

The total variation of temperature to be provided for in calculating the stresses in arches shall be according to the following formula:

$$T = 120 - 10d$$

if the spandrels are not filled with earth and

$$T = 120 - 15d$$

if the earth filling on the crown of the arch is not less than "d," in which

T = the total variation of temperature and
d = crown thickness of the arch.

Bearing Power of Piles.

The safe bearing power of piles, whether of concrete or of wood, shall be assumed to be in pounds equal to—

$$\frac{2WH}{S + 1}$$

in which

W = weight of hammer in pounds.
H = height of fall of hammer in feet.
S = penetration in inches per blow, average of last few blows.

provided the weight of the pile is less than 50 per cent of that of the hammer. If, however, the pile, with follower, cap, etc., weighs more than 50 per cent as much as the hammer, the bearing value shall be assumed to be 25 per cent less than that given by the above formula. If concrete piles are molded in place, measures shall be taken to prevent damage to them by the driving of neighboring piles or otherwise.

Bearing Power of Soils.

The following may be considered as the safe bearing power of soils per square foot:

MATERIAL.	BEARING POWER, TONS PER SQ. FT.
Quicksand and wet soils	0.1 to 1.0
Dry earth, according to depth below surface	1.0 to 3.0
Moderately dry clay, confined	2.0 to 4.0
Dry stiff clay	4.0 to 6.0
Sand, confined	2.0 to 6.0
Sand, compact and cemented	4.0 to 8.0
Gravel, cemented	8.0 to 12.0
Rock	25.0 to 200.0

Earth Pressure.

The pressure of earth against abutments and wing walls which are thoroughly drained and where the filling is well compacted shall be considered equal to that of a liquid weighing 30 lb. per cu. ft., where no pressure due to landslides is to be feared, otherwise existing conditions must be provided for.

Required Waterways

The waterway required for culverts may be computed by the following formula:

$$a = C A^{\frac{3}{4}}$$

in which

a = required area of the culvert in sq. ft.
A = drainage area in acres.
C = a constant depending on length and character of the drainage area, and may vary from about 0.3 to 2.0 in regions where the mean annual rainfall does not exceed 50 in. For a greater rainfall increase the value of C in proportion.

Removal of Forms.

Forms may be removed from the sides and tops of concrete to permit finishing as soon as the concrete has set sufficiently to remain firmly in place, which, in warm weather, may be from 24 to 48 hours, and longer in colder weather. Supports must be left under slabs and arches until the concrete has set sufficiently so that cubes of the same concrete will resist without deformation a compressive load of at least 1,000 lb. per sq. in. which in general will be about 20 days in warm weather and longer in cool weather.

Impact.

With the loading and unit stresses assumed in these specifications no allowance for impact will be necessary.

Respectfully submitted,

WILLIS WHITED,
Chairman.

DISCUSSION.

Mr. Willis Whited.—Mr. President, the work of this committee has been rather handicapped by the fact that it is only lately that its membership has been completed, so that there has been very little opportunity for the committee to get together and talk things over. The best I could do was to prepare a tentative specification to present to this meeting for discussion. Mr. Whited.

I will read a communication I received from Mr. Quimby, who could not be present today.

Mr. Henry H. Quimby (*by letter*).—I regard 37½ per cent of the 30-day value of concrete as inadvisable to recommend for the allowable unit stress in concrete of either arches or slabs; 30 per cent is as high as I think we should stand for on extreme fibers in bending or eccentric thrust. Mr. Quimby.

I am in favor of modifying the assumed weights of earth filling and of brickwork to 110 and 125 lb. respectively.

Under "weight imposed" I think that earth filling should not be depended upon to sustain itself by arching. Also the "due regard for the friction between the particles" is too indefinite for a specification.

Under live loads I would prescribe the 16-ton engine or roller for all country bridges. I think that no bridge should be designed for less. For interurban cars a spacing of axles can be just as definitely prescribed as the weight.

The formula for lateral distribution of track loads does not seem to be clear to me. Perhaps there is a type error in it. The rules for the distribution of a concentrated load on a slab I presume was intended to have the word at the end of the third line "weight" instead of "width." I think that the allowance of 1½ times the thickness of arch sheeting for distribution distance might be increased to three times. Seven-tenths of the depth of earth filling seems to me too much for distribution of a concentrated load and that the base of the assumed cone should not be more than the vertical height.

I would apply the requirement of 30 lb. hydraulic pressure

Mr. Quimby. against retaining walls only to surcharged walls and let 25 lb. do for walls not surcharged.

I think that the waterway formula is very desirable, but should be made more definite. I have not at hand anything just in the line of this case, but will look it up.

Now I have freely suggested as you desired, but I must add that I have some doubt as to the propriety of this Association undertaking to father a specification for design. Such a thing seems to me to be in the province rather of a state legislature and of an engineering society like the American Railway Engineering and Maintenance of Way Association, whose specifications might be referred to as a standard, just as are those for materials of the American Society for Testing Materials.

Mr. Lovis. MR. A. M. LOVIS.—I wish to make a few suggestions as to what should go into a report of this character. I think that the tentative report covers the ground fairly well. For bridges of classes A and B, referred to in the report, it seems to me that it is doubtful whether we should assume a lesser live load for one class than for the other. It is also probable that we may have to provide for much heavier loads in the future; that is, for motor trucks or traction engines.

The method of calculating the dead load on arches would appear to be satisfactory; also the assumption in regard to the amount of dead load to reach an arch bridge when the fill is very high above it, is probably safe.

We have been constructing reinforced concrete bridges in Massachusetts for over ten years; I think we were among the pioneers in using concrete for bridges. Most of our bridges under 35-ft. span have been beam and slab bridges. The designs were made to conform to early formulas and we have found by experience that the error, if any, was on the safe side; we continue to design beams by this formula, and use a table based on it. We propose this winter to prepare a new table based on the later formulas prepared by the Joint Committee on Concrete and Reinforced Concrete. It might be wise to insert in a report of this kind tables as well as formulas. I believe that a table for class B and C bridges, based on the formula suggested would be of value to any engineer desiring to design bridges of that type. We might advise as to the type of bridge to be used for different widths and

spans; we might advise for what spans the arch should be used. The wisdom of including such information in the report should be determined by the committee. Mr. Lovis.

It would seem to me that it would be well to emphasize in this report the necessity for full and complete surveys and careful preliminary soundings, and so forth, to aid the designer.

Mr. Whited has given a formula for calculating the area of openings. A table might be inserted in this report which would give these areas.

Mr. Daniel B. Luten.—I am wondering whether this report is to be a text-book on design or a standard specification for the construction of bridges. It seems to me that it contains far more of information for the engineer than for the contractor and that a great deal has been omitted that should be included if it is to be a standard specification. Coffer-dams, centering, finish, railing and filling are met with in the construction of arch bridges, yet are not included. If it is simply a statement of good practice in design, then we ought to be extremely careful as to what we approve. There are numerous text-books which contain far more on the subject. The practice in arch design is continually changing. I am opposed to some of the suggestions made. There never yet has been any acknowledged authority who advocated 750 lb. per sq. in. for concrete in compression. That is the highest that I have ever heard recommended for arches, and is 50 per cent higher than the recommendation of the Joint Committee. Mr. Luten.

The allowance to be made for the arching of earth filling over an arch is a difficult one to determine and one that we cannot accurately determine. For example, suppose a series of arches on piers, under very heavy earth fill; what becomes of the extremely heavy earth pressure? Is it largely carried to the piers, not by the arches, but by the arching of the earth itself? Suppose that a railroad train rattles over that fill at high speed; will not the vibration cause the entire earth fill over the span to bear vertically upon that arch? I do not believe it is safe to design a railway arch for arching effect such as is recommended. While it is understood that this recommendation is made for highway bridges, there is the same doubt as to arching effect, but to a lesser degree.

A concrete bridge has the great advantage of being permanent. Then why design a concrete bridge for ten tons on a side

Mr. Luten. road and for sixteens tons on a main road? The side road may become a main road in ten or twenty years and then the bridge will need replacing because it is not properly designed for the heavier load. There are many sections of our country in which road rollers weigh fifteen or twenty tons at the present time, and 100 lb. per sq. ft. is too low a value for a concrete bridge. Moreover, the concrete bridge can be designed for a load of 150 lb. at a cost of 5 per cent more than for a load of 100 lb.

Interurban cars in Indiana now weigh 60 tons; 50 tons is not a safe specification for Indiana; 100 tons is usually specified. In the eastern states, as in Massachusetts, where stops must occur so frequently that there cannot be high speed traffic with heavy cars, 25 tons may be sufficient.

The distribution of concentrated loads upon arches is a question for serious discussion. It may be very well to permit it to be distributed according to the formula suggested, on one-fourth of the earth fill, but what about the abutments? Is it possible for an arch bridge to fail in a single intermediate section of such width? Surely not. A more desirable distribution on the piers at least would be a maximum in the middle and a minimum at the sides of the roadway width; in designing an abutment or pier the live load thrust may certainly be distributed over the entire length of the abutment, and not over that single section on which the load on the arch is supported. I do not mean to recommend any particular method of design in preference to the one suggested, and only mention these points to show that it is questionable whether we should recommend any methods of design. The design of a concrete bridge is a matter of expert engineering, and this Association does not and never can exercise the functions of an expert engineer.

In regard to temperature changes, there is of course the usual change considered by designers due to uniform change of the entire structure. But in the case of a very thick arch the change is less rapid at the interior than at the exterior. The exterior shell will rise in temperature rapidly, the interior remaining at a lower temperature, consequently the exterior shell will carry the load; and the stress condition from temperature in a heavy arch may be much more dangerous than in the light arch. Will such a formula then which deals with uniform changes only be accept-

able? Will it not be better to leave such matters for expert designers and for investigators to solve? Mr. Luten.

I question the propriety of recommending a formula for waterway, because waterway is usually to be determined by conditions that apply at the location, by examination of neighboring structures and by consulting the residents regarding high-water records. Of course, a formula is of great assistance, but it is by no means final.

MR. WHITED.—These specifications were rather hurriedly prepared and a good many matters were left out that I intended to cover. As to what Mr. Luten says about specifications for construction, that is already pretty well covered by reports of other committees of this Association. The question naturally arises as to whether it is best to prepare specifications or recommended practice; whether this should cover such matters as protecting a pavement against scour by the construction of an apron wall to prevent water getting under it; the height and strength of the arching required; the strength and height of curbs; the camber of the bridge; the drainage of abutments; the question of high culverts, that will be a very important question in the near future; and should this report include specifications for stone masonry the different kinds of work in connection with that and also cyclopean concrete. Mr. Whited.

Some of Mr. Luten's points are well taken, and the committee is certainly glad to have any suggestions; the committee will also be glad to hear from anyone who has criticisms, suggestions or new points to offer.

PRESIDENT HUMPHREY.—The committee has combined recommended practice and specifications. To comply with the established practice of this Association they should be separated. As its work progresses the committee will do this differentiating between what is recommended practice and what specifications. The President.

MR. L. J. MENSCH.—I agree with the committee that 750 lb. should be allowed for all the stresses. The European standard is 700 lb. for even bridges built of brick or stone laid in cement mortar. The temperature stresses are included, and also the stresses due to the shrinkage of the arch and movement of the abutments; the latter always move a little, and a tenth of an inch, or for a small bridge a twentieth of an inch, makes quite a difference in the stresses. Mr. Mensch.

Mr. Mensch.

The report should clearly state what the 750 lb. include; ordinarily 500 lb. is the European practice; if all stresses are included 750 lb. is used.

The earth fill on the arch is quite often figured by European designers according to a formula as a pressure on the arch; this pressure increases the negative moment and additional reinforcement should be provided near the upper surface of the arch.

I think it wise to allow only 25 lb. for the pressure of the earth fill on the abutments; this does not act at the beginning, but after a few years, when the earth settles, the pressure is very much greater.

Some tests made a year ago showed that the distribution of a concentrated load by the earth fill is very much greater than is usually assumed.

I recall a test of a plate 4 in. in thickness, loaded with a uniform load over 3 ft. and with a concentrated load on a 4-in. plate which showed nearly the same deflection for each condition of loading.

The earth fill is generally assumed (for at least 50 years) to act on an angle of 45 deg., so that the extension would be twice the height of the actual fill. I have never heard of any case where this gave trouble. The earth must, of course, be well rolled and not carelessly filled in.

Professor Shattuck.

PROFESSOR HAROLD B. SHATTUCK.—In speaking for the Pennsylvania State College, I can say that the college is very glad to cooperate with Mr. Whited in undertaking these experiments for determining the distribution of pressure produced by concentrated loads through earth filling on top of culverts and bridges.

The distribution of pressure, as found by mathematical analysis from the measured deflections of loaded plates, it seems to me, is based on certain assumptions, so that the results arrived at can be no more reliable than are the assumptions made in the beginning, and on this account we have attempted to devise a mechanism whereby it will be possible to measure directly the actual pressure existing at different points more or less remote from the point of application of the concentrated load.

Our experiments have not yet gone far enough to give us any definite results. Our work has been mostly in revising and

adjusting our machine, but we hope to be able to find some clue to the solution of this problem. Professor Shattuck.

I came here with the intention not of giving you very much information in the matter, but of acquiring information myself, and if any members of the Association have suggestions to offer I am sure we at the Pennsylvania State College would be very glad to have them. That is about all I can say in regard to the matter at present. I hope at some future time to be able to give you something more definite.

MR. LUTEN.—Mr. President, permit me to add a suggestion. Mr. Luten. Lintels for doors and windows have always been figured as carrying only the brick-work in the triangle above them; there is an arching effect that carries the remainder of the material, but before such arching can relieve the lintel, there must be a yielding of the lintel beneath its load. The same thing is true of an arch. Before there can be any arching effect of the earth over the arch, the arch itself must show considerable settlement. This should be taken into consideration in any tests that are made.

I am strongly opposed to making this specification a text-book on design. It seems to me that is proposed by Mr. Mensch. If we specify 750 lb. as good practice, let us not even say that that includes temperature; say, instead, that 750 lb. is the allowable stress. Every engineer knows that that includes everything.

One of the worst faults that needs correction in concrete bridge practice is the bidding below cost of the unskilled contractor. Would it not be a good idea to include in every set of specifications a complete list of all the items that go to make up a bridge? I have here, for example, an estimate sheet for a concrete bridge which shows upwards of fifty separate items. This would do much to correct some of the faults of concrete bridge construction.

PROFESSOR A. N. TALBOT.—Mr. President, I have little to suggest at this time. Professor Talbot. It seems that it would be well for the committee to make a distinction between the compressive stress allowable in the flexure of slabs and that in abutments or arches. In tests there is found considerable difference in the ultimate stress in beam action and in direct compression. The load which will be carried by a beam before it will fail in compression is considerably higher than would appear from calculated stresses. If

Professor Talbot. the fiber stress in a beam is calculated to be 750 lb. per sq. in., the factor of safety for compression failure would be more than the ratio of 2,000 to 750, assuming concrete of 2,000 lb. per sq. in. compressive strength. In other words, we are assuming methods of calculation which are not exact. We are doing that for simplicity's sake; and we therefore have the right to make our working stresses accordingly.

In the case of direct compression, the conditions, as in the case of the arch ring, are much nearer pure compression than in beam action. It would seem then that the working compressive stress in arches should be lower than the permissible stress for beam action.

In the case of concentrated loads on slabs, I should say that the effect of the load and the resulting fiber stresses are distributed more widely than was indicated by Mr. Whited, much more widely laterally. There must be a considerable distribution of stress because of the necessity of having a fiber of the beam at one point stretch or compress nearly as much as the fiber next to it. That means that there will be a wide distribution of the stresses making up the resisting moment. What we are after, of course, is to take such a width of slab that the calculations will give the maximum stresses immediately under the load.

The stress in the concrete in the arch will have a wide lateral distribution, particularly, as Mr. Luten has suggested, towards the abutments, and perhaps we shall have to take into consideration this particular feature.

So far as dead loads are concerned, like earth, it would seem to me that while there is considerable distribution, the area given is larger than may be expected for the average load distribution. In other words, even if the load is distributed over as large an area as given by the ratio cited, that near the edge will be small and near the middle will be much larger than the average. Such an area should be chosen that by dividing the loads by it the result will give the maximum pressure at a certain point in the area.

In the matter of waterway, it would seem to me not wise to make anything very definite or explicit, since conditions vary so much over the country, and even according to the special location. A valuable paper on the subject of size of waterways may be found in the Proceedings of the American Railway Engineering and Main-

tenance of Way Association for 1910, in the report of the Committee on Roadways. Professor Talbot.

Mr. Whited.—Another question that came up in the preparation of this specification was whether it was advisable to include in it recommendations regarding the form of contract. I think it would be well for some association having the weight of this to take that matter up. But we decided against it for fear that it would prolong the discussion so much that we could not get results for a long time. Mr. Whited.

As to the effect of the 750-lb. load, I had some doubts, but wanted to get some expression from the members on this matter. I think for ordinary cases it is excessive and ought never to be applied unless every other stress is taken into consideration.

The effect of the earth is the maximum under the load and diminishes gradually to a certain minimum. The matter is so complex that it is difficult to get results, and we were in hope of getting some experimental results that would be a guide to engineers in working it out.

Now as to the distribution in slabs, I think with Professor Talbot that it is much wider than what Mr. Quimby recommends. I think that it is a function of the span as well as of the width; the distribution is quite wide; in making calculations we would have to take account of the width of the distribution instead of taking the total length of the distribution. That is to say, the load carried by adjacent portions of the slab diminished at first quite slowly and afterwards much more rapidly to zero at a certain distance; if it is as to the square of the distance, for instance, we would have to take a distribution of two-thirds of the total.

Mr. E. S. Larned.—I have one suggestion to make in connection with the proposed specification. It is not uncommon to find in approved specifications for building construction, road construction and a wide range of municipal work, a provision requiring the contractor to assume the full responsibility for the infringement of all patents on materials or devices used in, or in connection with, the particular work; the fact that this is so generally done would suggest that there is a good reason for it. Much has been said in regard to patents applying to bridge construction. It seems to me that if there is any warrant for the introduction of a clause of the nature referred to in ordinary muni- Mr. Larned.

Mr. Larned. cipal work, or in building construction, it would be an equally wise provision to incorporate the same in specifications for bridge construction until at least that much-mooted question is fully settled and better understood.

Mr. Whited. MR. WHITED.—It might be wise to put something in about patents, but we hesitated to do this. Specifications usually place the responsibility for patent infringement on the contractor; I think it is generally wise to do this. The contractor can protect himself by purchase or by acquiring a license, in which case everybody is protected. There is usually trouble when the purchaser attempts to protect the contractor in such matters.

REPORT OF THE COMMITTEE ON ROADWAYS, SIDEWALKS AND FLOORS.

The following revisions of the Standard Specifications are recommended:

Standard Specifications for Plain Concrete Floors.

Page 3. Paragraph 20. Last line. Change "one-half ($\frac{1}{2}$)" to read "three-eighths ($\frac{3}{8}$)."

Page 6. Paragraph 34. Line 2. Change "one and one-half ($1\frac{1}{2}$) part" to read "not more than two (2) parts."

Paragraph 36. Line 2. Change "two (2) inches from the finished surface of the floor" to read "in the middle of the section."

Paragraph 37. Line 3. Change "four (4)" to read "six (6)."

Standard Specifications for Reinforced Concrete Floors.

Page 5. Paragraph 24. Line 3. Change "four (4)" to read "six (6)."

Specifications for Concrete Curb and Concrete Curb and Gutter.

Page 7. Paragraph 32. Last line. Change "one-half ($\frac{1}{2}$)" to read "three-eighths ($\frac{3}{8}$)."

Paragraph 34. Line 2. Change "one and one-half ($1\frac{1}{2}$)" to read "not more than two (2)."

Paragraph 36. Change to read as follows:

36. *Treatment.*—As soon the concrete has hardened sufficiently to prevent being pitted, it shall be sprinkled with clean water and kept wet for at least four (4) days. The work shall not be opened to traffic until the engineer so directs.

Standard Specifications for Portland Cement Sidewalks.

Page 6. Paragraph 38. Change to read as follows:

38. *Treatment.*—As soon as the concrete has hardened sufficiently to prevent being pitted, the surface of the walk shall

be sprinkled with clean water and kept wet for at least four (4) days. The walk shall not be opened to traffic until the engineer so directs.

STANDARD SPECIFICATIONS FOR CONCRETE ROADS AND STREET PAVEMENTS.

Page 2. Paragraph 7. Line 3. Add a new sentence to read as follows:

Expansion Joints may also be formed by inserting during construction and leaving in place, a total thickness of one-quarter ($\frac{1}{4}$) inch of tarred paper or tarred felt.

Note at bottom of page. Line 2. Change "paragraphs 3 and 9 (*a*)" to read "paragraphs 3, 9 (*a*), 13 and 14."

Page 3. Paragraph 18. Line 1. Change "One-half ($\frac{1}{2}$) inch expansion joints" to read "Expansion Joints not less than one-quarter ($\frac{1}{4}$) inch nor more than one-half inch in width."

Line 4. Change "similar" to read "one-half ($\frac{1}{2}$) inch."

Paragraph 19. Line 4. Change "one-half ($\frac{1}{2}$)" to read "three-eighths ($\frac{3}{8}$)."

Page 4. Paragraph 25. Line 3. Change "individual sections" to read "sections between expansion joints without the use of intermediate cross forms or bulk heads."

Page 5. Paragraph 26. Line 1. Change "twenty-five (25)" to read "twenty (20)."

Paragraph 28. Line 3. Add after "position" the words "with a template or straight edge."

Paragraph 31. Lines 3–7. Change to read as follows: "manner that will thoroughly compact it and produce a comparatively smooth surface, free from depressions or inequalities of any kind. The finished surface of the concrete shall not vary more than one-quarter ($\frac{1}{4}$) inch from a two (2) foot straight edge placed upon the concrete in any position.

Page 6. Paragraph 33. Line 2. Change "one and one-half ($1\frac{1}{2}$)" to read "two (2)."

Paragraph 34. Lines 2–5. Change to read as follows: "provided for under Two-Course Pavement, base and wearing course respectively."

Paragraph 35. Line 2. Change "two (2) inches from the upper surface of the pavement" to read "at the middle of the section."

Page 7. Paragraph 41. Renumber to read "Paragraph 36."

Paragraph 42. Renumber to read "Paragraph 37" as follows:

37. *Treatment.*—As soon as the concrete has hardened sufficiently to prevent being pitted, the surface of the pavement shall be sprinkled with clean water and shall be kept wet for at least four (4) days. Concrete pavement on roads shall be covered as soon after finishing as it is possible to do so without damaging the surface, with at least two (2) inches of dirt, which shall be kept wet for at least four (4) days. Before covering with dirt, the pavement shall be sprinkled with water as above specified. The pavement shall not be open to traffic until the engineer so directs.

Paragraph 43. Renumber to read "Paragraph 38."

Insert a new Paragraph 39 to read as follows:

Shoulders.

39. *Construction.*—On streets where the pavement does not occupy the full width of the street, and on roads, a gravel or crushed stone shoulder at least two (2) feet wide shall be constructed on each side of the pavement. The surface of the shoulders shall have a slope away from the pavement of one and one-half ($1\frac{1}{2}$) inches per foot, and a thickness for the two (2) foot width adjoining the concrete at least equal to the minimum thickness of the concrete.

Page 6. Paragraph 36. Renumber to read "Paragraph 40" as follows:

40. *Construction.*—Where a wearing surface of bitumen and fine aggregate is used, it shall preferably be placed upon a one-

course pavement constructed as hereinbefore specified, but may be used also on two-course work.

Page 7. Paragraph 40. Renumber to read "Paragraph 41."
Line 1. Change "wearing surface" to read "bitumen."
Line 2. Change "of bitumen and sand" to read "to the concrete."
Change "all expansion joints in the pavement" to read "all open expansion joints."

Page 6. Paragraph 37. Renumber to read "Paragraph 42."
Paragraph 38. Renumber to read "Paragraph 43."
Line 2. Change "the clean dry" to read "the thoroughly clean dry."
Line 6. Strike out the words "immediately after applying."
Paragraph 39. Renumber to read "Paragraph 44."
Line 1. Change "Between one-third ($\frac{1}{3}$) and" to read "approximately."

Respectfully submitted,

C. W. BOYNTON, *Chairman*,
A. C. BIRNIE,
J. B. LANDFIELD,
C. R. MILLER,
A. E. SNODGRASS.

National Association of Cement Users.

HARRISON BUILDING,
PHILADELPHIA, PA.

STANDARD SPECIFICATIONS FOR PLAIN CONCRETE FLOORS.

MATERIALS.

Cement.

1. The cement shall meet the requirements of the Standard Specifications for Portland Cement of the American Society for Testing Materials and adopted by this Association (Standard No. 1).

Aggregates.

2. The aggregates shall be clean, coarse, hard, durable materials and shall be free from dust, soft, flat or elongated particles, loam, vegetable or other deleterious matter. In no case shall aggregate containing frost or lumps of frozen material be used.

(*a*) *Fine Aggregate for Concrete.*—Fine aggregate shall consist of sand, crushed stone or gravel screenings, preferably of silicious material, graded from fine to coarse and passing, when dry, a screen having one-quarter ($\frac{1}{4}$) in. diameter holes, and not more than three (3) per cent passing a sieve having one hundred (100) meshes per lin. in.

Fine aggregate shall be of such quality that mortar composed of one (1) part Portland cement and three (3) parts fine aggregate by weight when made into briquettes will show a tensile strength at least equal to the strength of 1 : 3 mortar of the same consistency made with the same cement and Standard Ottawa sand.

(*b*) *Aggregate for Wearing Course.*—The aggregate shall consist of screened gravel or stone screenings from granite or other close-grained, durable rock, sufficiently hard to scratch glass, mixed in the proportion of three (3) parts passing a one-half ($\frac{1}{2}$) in. ring and retained on a screen having one-quarter ($\frac{1}{4}$) in. diameter holes, and two (2) parts passing a screen having one-quarter ($\frac{1}{4}$) in. diameter holes and retained on a screen having fifty (50) meshes per lin. in.

(*c*) *Coarse Aggregate for Concrete.*—Coarse aggregate shall consist of inert materials, such as crushed stone or gravel, graded in size, retained on a screen having one-quarter ($\frac{1}{4}$) in. diameter holes and the maximum size shall be such as to pass a one and one-quarter ($1\frac{1}{4}$) in. ring.

(*d*) *Natural Mixed Aggregates.*—Natural mixed aggregates shall not be used as they come from the deposit but shall be screened and remixed to agree with the proportions specified.

Sub-Base. 3. Only clean, hard, suitable material, not exceeding four (4) in. in the largest dimension shall be used in the sub-base.*

Water. 4. Water shall be clean, free from oil, acid, alkali or vegetable matter.

Color. 5. If artificial coloring material is required, only mineral colors shall be used.

Reinforcement. 6. The reinforcing metal shall meet the requirements of the Standard Specifications for Steel Reinforcement adopted March 16, 1910, by the American Railway Engineering Association.

Joint Filler. 7. The joint filler shall be a suitable elastic waterproof compound that will not become soft and run out in hot weather, nor hard and brittle and chip out in cold weather.

SUB-GRADE.*

Sub-Grade. 8. *Depth.*—(*a*) The sub-grade shall not be less than eleven (11) in. below the finished surface of the floor.

(*b*) The sub-grade shall not be less than five (5) in. below the finished surface of the floor.

9. *Preparation.*—All soft and spongy places shall be removed and all depressions filled with suitable material which shall be thoroughly compacted in layers not exceeding six (6) in. in thickness.

10. *Deep Fills.*—When a fill exceeeding one (1) ft. in thickness is required to bring the work to grade, it shall be made in a manner satisfactory to the engineer.

Drainage. 11. *Drainage.*—When required, a suitable drainage system shall be installed and connected with sewers or other drains indicated by the engineer.

---

* When a sub-base is required, eliminate paragraph 8 (*b*). When a sub-base is not required, eliminate paragraphs 3 and 8 (*a*). Unless paragraph 8 (*a*) is eliminated, 8 (*b*) is void.

SUB-BASE.

12. *Thickness.*—On the sub-grade shall be spread a material as hereinbefore specified, which shall be thoroughly rolled or tamped to a surface at least five (5) in. below the finished grade of the floor. Sub-Base.

13. *Wetting.*—While compacting the sub-base, the material shall be kept thoroughly wet, and shall be in that condition when the concrete is deposited.

FORMS.

14. *Materials.*—Forms shall be free from warp and of sufficient strength to resist springing out of shape. Forms.

15. *Setting.*—The forms shall be well staked or otherwise held to the established lines and grades and their upper edges shall conform to the established grade of the floor.

16. *Treatment.*—All wood forms shall be thoroughly wetted and metal forms oiled before depositing any material against them. All mortar and dirt shall be removed from forms that have been previously used.

CONSTRUCTION.

17. *Size of Slabs.*—The slabs or independently divided blocks when not reinforced shall have an area of not more than one hundred (100) sq. ft., and shall not have any dimension greater than ten (10) ft. Larger slabs shall be reinforced as hereinafter specified. Slabs.

18. *Thickness of Floor.*—The thickness of the floor shall not be less than five (5) in.

19. *Width and Location of Joints.*—When required by the engineer in charge a one-half ($\frac{1}{2}$) in. space or joint shall be left between the floor and the walls and columns of the building. Joints.

20. *Protection of Edges.*—Where required by the engineer in charge, the joints shall be protected by metal. Unless protected by metal, the upper edges of the concrete shall be rounded to a radius of three-eighths ($\frac{3}{8}$) in. Edges.

MEASURING AND MIXING.

21. *Measuring.*—The method of measuring the materials for the concrete, including water, shall be one which will insure Measuring.

separate uniform proportions at all times. A bag of Portland cement (94 lb. net) shall be considered one (1) cu. ft.

22. The ingredients of the concrete or mortar shall be thoroughly mixed dry, sufficient water added to obtain the desired consistency, and the mixing continued until the materials are uniformly distributed and the mass is uniform in color and homogeneous.

Mixing.

(*a*) *Machine Mixing.*—When the condition will permit, a machine mixer of a type which insures the uniform proportioning of the materials throughout the mass, shall be used.

(*b*) *Hand Mixing.*—When it is necessary to mix by hand, the mixing shall be on a water-tight platform and the materials shall be turned until the mass is uniform in color and homogeneous.

Retempering.

23. *Retempering*, that is, remixing mortar or concrete that has partially hardened with additional water, will not be permitted.

TWO-COURSE FLOORS.

Two-Course Floor. Base.

24. *Proportion.*—The concrete shall be mixed in the proportion by volume of one (1) part Portland cement, two and one-half ($2\frac{1}{2}$) parts fine aggregate, and five (5) parts coarse aggregate.

25. *Consistency.*—The materials shall be mixed wet enough to produce a concrete of a consistency that will flush readily under slight tamping, but which can be handled without causing a separation of the coarse aggregate from the mortar.

26. *Placing.*—After mixing, the concrete shall be handled rapidly and the successive batches deposited in a continuous operation completing individual sections. Under no circumstances shall concrete be used that has partially hardened. The forms shall be filled and the concrete struck off and tamped to a surface the thickness of the wearing course below the established grade of the floor. After the concrete has been thoroughly tamped against the cross forms they shall be removed and the material for the adjoining slab deposited so as to preserve the joint. Workmen shall not be permitted to walk on freshly-laid concrete, and if sand or dust collects on the base it shall be carefully removed before the wearing course is applied.

Reinforcement.

27. Slabs having an area of more than one hundred (100) sq. ft., or having any dimension greater than ten (10) ft., shall be

reinforced with wire fabric, or with plain or deformed bars. The cross-sectional area of metal shall amount to at least 0.041 sq. in. per lin. ft. The reinforcement shall be placed upon and slightly pressed into the concrete base immediately after the base is placed and shall not cross joints and shall be lapped sufficiently to develop the strength of the metal.

Wearing Course.

28. *Proportions.*—The mortar shall be mixed in the manner hereinbefore specified in the proportion by volume of one (1) part Portland cement, and not more than two (2) parts aggregate for wearing course.

29. *Consistency.*—The mortar shall be of a consistency that water will flush to the surface only under heavy, thorough working with a wood float.

30. *Thickness.*—The wearing course of the floor shall have a minimum thickness of one (1) in.

31. *Placing.*—The wearing course shall be placed immediately after mixing and in no case shall more than fifty (50) minutes elapse between the time the concrete for the base is mixed and the time the wearing course is placed.

32. *Finishing.*—After the wearing course has been brought to the established grade, it shall be worked with a wood float in a manner which will thoroughly compact it. When required, the surface shall be troweled smooth, but excessive working with a steel trowel shall be avoided. The slab markings shall be made in the wearing course directly over the joints in the base with a tool which will completely separate the wearing courses of adjacent slabs. If excessive moisture occurs on the surface, it must be taken up with a rag or mop and in no case shall dry cement or a mixture of dry cement and sand be used to absorb this moisture or hasten the hardening. Unless protected by metal the surface edges of all floor slabs shall be rounded to a radius of about one-half ($\frac{1}{2}$) in.

33. *Coloring.*—If artificial coloring is used, it must be incorporated with the entire wearing course and shall be mixed dry with the cement and aggregate until the mixture is of uniform color. In no case shall the amount of coloring used exceed five (5) per cent of the weight of the cement.

ONE-COURSE FLOOR.

One-Course. Floor.

The general requirements of the specifications covering two-course work will apply to one-course work, with the following exceptions:

34. *Proportions.*—The concrete shall be mixed in the proportion by volume of one (1) part Portland cement to not more than two (2) parts fine aggregate (Paragraph 2 *a*) or aggregate for wearing course (Paragraph 2 *b*) and three (3) parts coarse aggregate passing a one (1) in. ring.

35. *Placing and Finishing.*—The forms shall be filled, the concrete struck off and the coarse particles forced back from the surface, and the work finished in the usual way.

36. *Reinforcement.*—When a one-course floor is to be reinforced, the metal shall be placed in the middle of the section. The minimum amount of metal shall be as specified in Paragraph 27.

PROTECTION.

Protection.

37. *Treatment.*—When completed, the floor shall be covered with at least two (2) in. of wet sand or sawdust which shall be kept moist for six (6) days and no one shall be allowed on the floor during that time.

38. *Temperature below 35° F.*—If at any time during the progress of the work the temperature is or, in the opinion of the engineer, will within twenty-four (24) hours drop to 35° F., the water and aggregate shall be heated and precaution taken to protect the work from freezing for at least five days. In no case shall concrete be deposited upon a frozen sub-grade or sub-base.

National Association of Cement Users.

HARRISON BUILDING,
PHILADELPHIA, PA.

STANDARD SPECIFICATIONS FOR REINFORCED CONCRETE FLOORS.

MATERIALS.

1. The cement shall meet the requirements of the Standard Specifications for Portland Cement of the American Society for Testing Materials and adopted by this Association (Standard No. 1). Cement.

2. The aggregates shall be clean, coarse, hard, durable materials and shall be free from dust, soft, flat or elongated particles, loam, vegetable or other deleterious matter. In no case shall aggregate containing frost or lumps of frozen material be used. Aggregates.

(*a*) *Fine Aggregate for Concrete.*—Fine aggregate shall consist of sand, crushed stone or gravel screenings, preferably of silicious material, graded from fine to coarse and passing, when dry, a screen having one-quarter ($\frac{1}{4}$) in. diameter holes, and not more than three (3) per cent shall pass a sieve having one hundred (100) meshes per lin. in.

Fine aggregate shall be of such quality that mortar composed of one (1) part Portland cement and three (3) parts fine aggregate, by weight, when made into briquettes will show a tensile strength at least equal to the strength of 1 : 3 mortar of the same consistency made with the same cement and Standard Ottawa sand.

(*b*) *Aggregate for Wearing Course.*—The aggregate shall consist of screened gravel or stone screenings from granite or other close-grained durable rock, sufficiently hard to scratch glass, mixed in the proportion of three (3) parts passing a one-half ($\frac{1}{2}$) in. ring and retained on a screen having one-quarter ($\frac{1}{4}$) in. diameter holes, and two (2) parts passing a screen having one-quarter ($\frac{1}{4}$) in. diameter holes and retained on a screen having fifty (50) meshes per lin. in.

(*c*) *Coarse Aggregate for Concrete.*—Coarse aggregate shall consist of inert materials, such as crushed stone or gravel, graded in size, retained on a screen having one-quarter ($\frac{1}{4}$) in. diameter holes, and the maximum size shall be such as to pass a one and one-quarter ($1\frac{1}{4}$) in. ring.

(*d*) *Natural Mixed Aggregates.*—Natural mixed aggregates shall not be used as they come from the deposit but shall be screened and remixed to agree with the proportions specified.

Water. 3. Water shall be clean, free from oil, acid, alkali, or vegetable matter.

Coloring. 4. If artificial coloring material is required, only mineral colors shall be used.

Reinforcement. 5. The reinforcement shall meet the requirements of the Standard Specifications for Steel Reinforcement adopted March 16, 1910, by the American Railway Engineering Association.

CONSTRUCTION.

Forms. 6. The forms shall be substantial, unyielding and so constructed that the concrete will conform to the designed dimensions and contours and shall also be tight to prevent the leakage of mortar. The supports for floors shall not be removed in less than ten (10) days after the concrete is placed, and then only with the consent of the engineer in charge.

Reinforcement. 7. The reinforcement shall be free from rust, scale or coatings of any character which will tend to reduce or destroy the bond, and shall be placed and held in position so that it will not become disarranged during the depositing of the concrete. Whenever it is necessary to splice tension reinforcement, the character of the splice shall be such as will develop its full strength. Splices at the point of maximum stress shall be avoided.

MEASURING AND MIXING.

Measuring. 8. The method of measuring the materials for the concrete, including water, shall be one which will insure separate uniform proportions at all times. A bag of Portland cement (94 lb. net) shall be considered one (1) cu. ft.

Mixing. 9. The ingredients of the concrete or mortar shall be thoroughly mixed dry, sufficient water added to obtain the desired

consistency and the mixing continued until the materials are uniformly distributed and the mass is uniform in color and homogeneous.

(*a*) *Machine Mixing.*—When the conditions will permit, a machine mixer of a type which insures the uniform proportioning of the materials throughout the mass, shall be used.

(*b*) *Hand Mixing.*—When it is necessary to mix by hand, the mixing shall be on a water-tight platform and the materials shall be turned until the mass is uniform in color and homogeneous.

Retempering.

10. *Retempering,* that is, remixing mortar or concrete that has partially hardened with additional water, will not be permitted.

Two-Course Floors.

Slabs.

11. *Proportions.*—The concrete shall be mixed in the proportion by volume of one (1) part Portland cement, two (2) parts fine aggregate, and four (4) parts coarse aggregate.

12. *Consistency.*—The materials shall be mixed wet enough to produce a concrete of a consistency that will flow into the forms and about the reinforcement, but which can be conveyed from the mixer to the forms without the separation of the coarse aggregate from the mortar.

13. *Placing.*—When it is necessary to make a joint in a floor slab, its location shall be designated by the engineer. The concrete shall be placed in a manner to insure a smooth ceiling, and thoroughly worked around the reinforcement and into the recesses of the forms. It shall be brought to a surface at least one (1) in. below the finished surface of the floor and as soon as the slab has hardened sufficiently to permit of workmen being upon it, and before it has dried out, the wearing course shall be applied. Workmen shall not be permitted to walk on freshly laid concrete, and if sand or dust collects on the base, it shall be carefully removed before the wearing course is applied.

Wearing Course.

14. *Proportions.*—The mortar shall be mixed in the manner hereinbefore specified in the proportion of one (1) part Portland cement and not more than two (2) parts aggregate for wearing course.

15. *Consistency.*—The mortar shall be of a consistency that

water will flush to the surface only under heavy, thorough working with a wood float.

16. *Thickness.*—The wearing course of the floor shall have a minimum thickness of one (1) in.

17. *Placing.*—The wearing course shall be placed immediately after mixing.

18. *Finishing.*—After the wearing course has been brought to the established grade, it shall be worked with a wood float in a manner which will thoroughly compact it. When required, the surface shall be troweled smooth, but excessive working with a steel trowel shall be avoided. If excessive moisture occurs on the surface, it must be taken up with a rag or mop, and in no case shall dry cement or a mixture of dry cement and sand be used to absorb this moisture or to hasten the hardening.

19. *Coloring.*—If artificial coloring is used, it must be incorporated with the entire wearing course and shall be mixed dry with the cement and aggregate until the mixture is of uniform color. In no case shall the amount of coloring exceed five (5) per cent of the weight of the cement.

WEARING SURFACE ON HARDENED SLABS.

Preparation of Slab.

The general requirements of the specifications covering two-course work will apply to Wearing Surface on Hardened Slabs with the following exceptions:

20. All dirt and débris shall be removed from the surface of the slab and the surface roughened in a manner that will expose the freshly broken face of the coarse aggregate. The floor shall be thoroughly scrubbed with a warm soap, lye, or acid solution, and rinsed with clear water, preferably under pressure. All water used to clean the slab shall be taken up with a rag or mop or otherwise disposed of.

21. *Treatment of Slab.*—The slab shall be thoroughly wet at the time the wearing course is placed. The surface of the slab shall be carefully treated with a neat cement grout brushed on, or with such a preparation as the engineer may direct.

Wearing Course.

22. *Consistency.*—The mortar for the lower one-quarter ($\frac{1}{4}$) in. shall be of such a consistency that it can be easily spread with a straight edge. The mortar for the remainder of the wearing surface shall be of the consistency specified for wearing surface.

23. *Placing.*—The wearing surface shall be placed in two operations. Upon the freshly placed cement grout, a one-quarter ($\frac{1}{4}$) in. layer of mortar of the consistency specified, shall be placed and thoroughly worked into the irregularities of the surface of the slab, and immediately followed with sufficient mortar of the consistency specified for wearing course to bring the floor to the established grade.

Protection.

24. *Treatment.*—When completed, the floor shall be covered with at least two (2) in. of wet sand or sawdust which shall be kept moist for six (6) days and no one shall be allowed on the floor during that time. Protection.

25. *Temperature below 35° F.*—If at any time during the progress of the work the temperature is or, in the opinion of the engineer, will within twenty-four (24) hours drop to 35° F., the water and aggregate shall be heated and precautions taken to protect the work from freezing for at least five days. In no case shall concrete be deposited upon a frozen base.

National Association of Cement Users.

HARRISON BUILDING,
PHILADELPHIA, PA.

STANDARD SPECIFICATIONS FOR CONCRETE CURB AND CONCRETE CURB AND GUTTER.

MATERIALS.

Cement. 1. The cement shall meet the requirements of the Standard Specifications for Portland Cement of the American Society for Testing Materials and adopted by this Association (Standard No. 1).

Aggregates. 2. The aggregates shall be clean, coarse, hard, durable materials and shall be free from dust, soft, flat or elongated particles, loam, vegetable or other deleterious matter and in no case shall aggregates containing frost or lumps of frozen material be used.

(*a*) *Fine Aggregate.*—Fine aggregate shall consist of sand, crushed stone or gravel screenings, preferably of silicious material, graded from fine to coarse and passing when dry a screen having one-quarter ($\frac{1}{4}$) in. diameter holes, and not more than three (3) per cent shall pass a sieve having one hundred (100) meshes per lin. in.

Fine aggregate shall be of such quality that mortar composed of one (1) part Portland cement and three (3) parts fine aggregate by weight when made into briquettes, will show a tensile strength at least equal to the strength of 1 : 3 mortar of the same consistency made with the same cement and Standard Ottawa sand.

(*b*) *Coarse Aggregate.*—Coarse aggregate shall consist of inert materials such as crushed stone or gravel graded in size, retained on a screen having one-quarter ($\frac{1}{4}$) in. diameter holes, and the maximum size shall be such as to pass a one and one-quarter ($1\frac{1}{4}$) in. ring.

(*c*) *Natural Mixed Aggregates.*—Natural mixed aggregates shall not be used as they come from the deposit, but shall be screened and remixed to agree with the proportions specified.

3. Only clean, hard, suitable materials, not exceeding four (4) in. in the largest dimension shall be used in the sub-base. **Sub-Base.**

4. Water shall be clean, free from oil, acid, alkali, or vegetable matter. **Water.**

5. If artificial coloring material is required, only mineral colors shall be used. **Coloring.**

6. The expansion joint filler shall be a suitable, elastic, waterproof compound that will not become soft and run out in hot weather, nor hard and brittle and chip out in cold weather. **Joint Filler.**

SUB-GRADE.

7. *Depth below Grade.* (*a*) *Concrete Curb.*—When a sub-base is required, the sub-grade shall not be less than thirty (30) in. below the established grade of the curb. **Depth.**

(*b*) *Concrete Curb and Gutter.*—When a sub-base is required, the sub-grade shall not be less than eleven (11) in. below the established grade of the gutter.

8. *Preparation.*—All soft and spongy places shall be removed and all depressions filled with suitable material which shall be thoroughly compacted in layers not exceeding six (6) in. in thickness. **Fills.**

9. *Deep Fills.*—When a fill exceeding one (1) ft. in thickness is required to bring the work to grade, it shall be made in a manner satisfactory to the engineer.

10. *Drainage.*—When required, a suitable drainage system shall be installed and connected with sewers or other drains indicated by the engineer. **Drainage.**

SUB-BASE.

11. *Thickness.* (*a*) *Concrete Curb.*—On the sub-grade shall be spread a material as heretofore specified, which shall be thoroughly rolled or tamped to a surface at least twenty-four (24) in. below the established grade of the curb. **Thickness.**

(*b*) *Concrete Curb and Gutter.*—On the sub-grade shall be spread a material as heretofore specified, which shall be thoroughly rolled or tamped to a surface at least six (6) in. below the established grade of the gutter.

12. *Wetting.*—While compacting the sub-base, the material shall be kept thoroughly wet and shall be in that condition when the concrete is deposited.

FORMS.

Forms.

13. *Materials.*—Forms shall be free from warp and of sufficient strength to resist springing out of shape.

14. *Setting.*—The forms shall be well staked or otherwise held to the established lines and grades, and their upper edges shall conform to the established grade of the curb or curb and gutter.

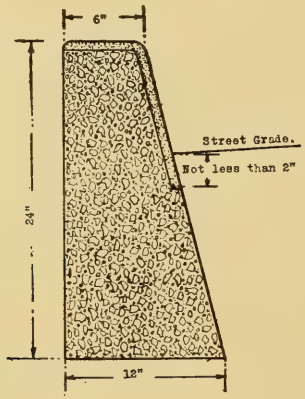


FIG. 1.

15. *Treatment.*—All wood forms shall be thoroughly wetted and metal forms oiled before depositing any material against them. All mortar and dirt shall be removed from forms that have been previously used.

CONSTRUCTION.

Curb.

16. *Dimension of Curb.*—The section of the curb shall conform with that shown in Fig. 1. The thickness at the base shall not be less than twelve (12) in., and at the top not more than six (6) in. with a batter on the street side of one (1) to four (4).

Curb and Gutter.

17. *Dimensions of Curb and Gutter.*—The sections of the combination curb and gutter shall conform with that shown in Fig. 2. The depth of the back of the curb shall not be less than

twelve (12) in. and the depth of the face not less than six (6) in. The breadth of the gutter shall not be less than sixteen (16) in. nor more than twenty-four (24) in.

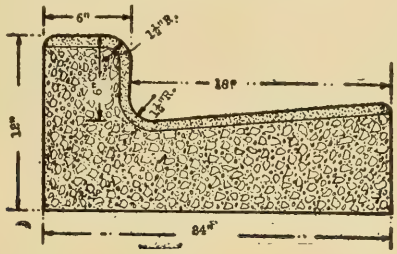


FIG. 2.

18. *Size of Sections.*—The curb and gutter shall be divided into sections not less than five (5) nor more than eight (8) ft. long by some method which will insure the complete separation of the sections. Sections

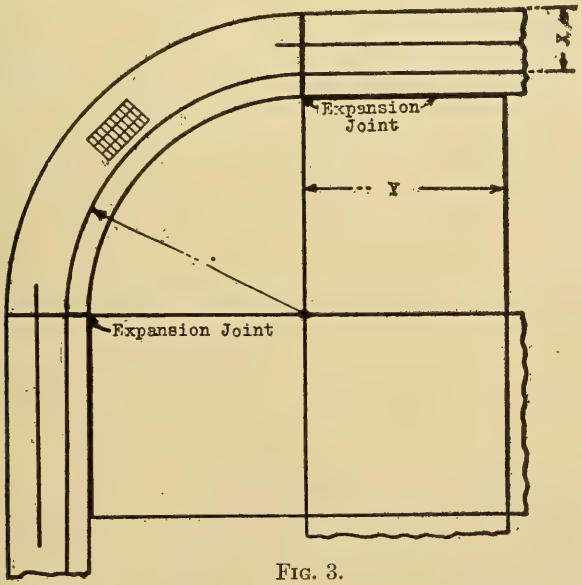


FIG. 3.

19. *Section at Street Corners.*—The construction of the combination curb and gutter at street corners shall conform with that

shown in Fig. 3. The radius of the curb shall not be less than six (6) ft.

Joints. 20. *Width and Location of Joints.*—A one-half ($\frac{1}{2}$) in. expansion joint shall be provided at least once in every one hundred and fifty (150) ft.

Edges. 21. *Protection of Edges.*—Unless protected by metal, the upper edges of the concrete shall be rounded to a radius of one-half ($\frac{1}{2}$) in.

MEASURING AND MIXING.

Measuring. 22. *Measuring.*—The method of measuring the materials for the concrete, including water, shall be one which will insure separate uniform proportions at all times. A bag of Portland cement (94 lb. net) shall be considered one (1) cu. ft.

Mixing. 23. The ingredients of the concrete or mortar shall be thoroughly mixed dry, sufficient water added to obtain the desired consistency, and the mixing continued until the materials are uniformly distributed and the mass is uniform in color and homogeneous.

(*a*) *Machine Mixing.*—When the conditions will permit, a machine mixer of a type which insures the uniform proportioning of the materials throughout the mass shall be used.

(*b*) *Hand Mixing.*—When it is necessary to mix by hand, the mixing shall be on a water-tight platform and the materials shall be turned until the mass is uniform in color and homogeneous.

Retempering. 24. Retempering, that is, remixing mortar or concrete that has partially hardened with additional water, will not be permitted.

TWO-COURSE CURB AND CURB AND GUTTER.

BASE.

Two-Course Work. Base. 25. *Proportions.*—The concrete shall be mixed in the proportion by volume of one (1) part Portland cement, two and one-half ($2\frac{1}{2}$) parts fine aggregate, and five (5) parts coarse aggregate.

26. *Consistency.*—The materials shall be mixed wet enough to produce a concrete of a consistency that will flush readily under slight tamping, but which can be handled without causing a separation of the coarse aggregate from the mortar.

27. *Placing.*—After mixing, the concrete shall be handled rapidly and the successive batches deposited in continuous operation completing individual sections. Under no circumstances shall concrete be used that has partially hardened. The gutter forms shall be filled and the concrete struck off and tamped to a surface the thickness of the wearing course below the established grade of the gutter. The concrete for the curb shall be placed and tamped so as to permit of the application of the required wearing course to the face and top so as to bring the work to the established line and grade of the curb. The work shall be executed in a manner which will insure perfect joints between abutting sections. Workmen shall not be permitted to walk on freshly laid concrete, and if sand or dust collects on the base, it shall be carefully removed before the wearing course is applied.

WEARING COURSE.

28. *Proportions.*—The mortar shall be mixed in the manner hereinbefore specified in the proportion by volume of one (1) part Portland cement and not more than two (2) parts fine aggregate. **Wearing Course.**

29. *Consistency.*—The mortar shall be of a consistency that will not require tamping but which can be easily spread into position.

30. *Thickness.*—The wearing course of the gutter and top and face of the curb shall have a minimum thickness of three-quarters ($\frac{3}{4}$) of an in.

31. *Placing.*—The wearing course shall be placed immediately after mixing, and in no case shall more than fifty (50) minutes elapse between the time the concrete for the base is mixed and the time the wearing course is placed.

32. *Finishing.*—After the wearing course has been brought to the established line and grade, it shall be worked with a wood float in a manner which will thoroughly compact it. When required, the surface shall be troweled smooth, but excessive working with a steel trowel shall be avoided. The section markings shall be made in the wearing courses directly over the joints in the base with a tool which will completely separate the wearing courses of adjacent sections. If excessive moisture occurs on the surface, it must be taken up with a rag or mop, and in no case shall dry cement, or a mixture of dry cement and sand, be used

to absorb this moisture or to hasten the hardening. The edge of the curb on the street side and the intersection of the curb and gutter shall be rounded to a radius of about one and one-half ($1\frac{1}{2}$) in. All other edges shall be rounded to a radius of three-eighths ($\frac{3}{8}$) in. unless protected by metal.

Coloring.

33. If artificial coloring is used, it must be incorporated with the entire wearing course and shall be mixed dry with the cement and aggregate until the mixture is of uniform color. In no case shall the amount of coloring used exceed five (5) per cent of the weight of the cement.

ONE-COURSE CURB AND ONE-COURSE CURB AND GUTTER.

One-Course Work.

The general requirements of the specifications covering two-course work will apply to one-course work, with the following exceptions:

34. *Proportions.*—The concrete shall be mixed in the proportion by volume of one (1) part Portland cement and two (2) parts fine aggregate, and three (3) parts coarse aggregate passing a one (1) in. ring.

35. *Placing and Finishing.*—The forms shall be filled, the concrete struck off and the coarse particles forced back from the surface, and the work finished as specified under two-course work.

PROTECTION.

Protection.

36. *Treatment.*—As soon as the concrete has hardened sufficiently to prevent being pitted, it shall be sprinkled with clean water and kept wet for at least four (4) days. The work shall not be opened to traffic until the engineer so directs.

37. *Temperature below 35° F.*—If at any time during the progress of the work, the temperature is or, in the opinion of the engineer, will within twenty-four (24) hours drop to thirty-five degrees (35°) F., the water and aggregates shall be heated and precautions taken to protect the work from freezing for at least five (5) days. In no case shall concrete be deposited upon a frozen sub-grade or sub-base.

National Association of Cement Users.

HARRISON BUILDING,
PHILADELPHIA, PA.

STANDARD SPECIFICATIONS FOR PORTLAND CEMENT SIDEWALKS.

MATERIALS.

Cement. 1. The cement shall meet the requirements of the Standard Specifications for Portland Cement of the American Society for Testing Materials and adopted by this Association (Standard No. 1).

Aggregates. 2. The aggregates shall be clean, coarse, hard, durable materials and shall be free from dust, soft, flat or elongated particles, loam, vegetable or other deleterious matter. In no case shall aggregate containing frost or lumps of frozen material be used.

(*a*) *Fine Aggregate.*—Fine aggregate shall consist of sand, crushed stone or gravel screenings, preferably of silicious material, graded from fine to coarse and passing when dry, a screen having one-quarter ($\frac{1}{4}$) in. diameter holes, and not more than three (3) per cent shall pass a sieve having one hundred (100) meshes per lin. in.

Fine aggregate shall be of such quality that mortar composed of one (1) part Portland cement and three (3) parts fine aggregate by weight, when made into briquettes will show a tensile strength at least equal to the strength of 1 : 3 mortar of the same consistency made with the same cement and Standard Ottawa sand.

(*b*) *Coarse Aggregate.*—Coarse aggregate shall consist of inert materials such as crushed stone or gravel, graded in size, retained on a screen having one-quarter ($\frac{1}{4}$) in. diameter holes and the maximum size shall be such as to pass a one and one-quarter ($1\frac{1}{4}$) in. ring.

(*c*) *Natural Mixed Aggregates.*—Natural mixed aggregates shall not be used as they come from the deposit, but shall be screened and remixed to agree with the proportions specified.

Sub-Base. 3. Only clean, hard, suitable material, not exceeding four (4) in. in the largest dimension shall be used in the sub-base.*

Water. 4. Water shall be clean, free from oil, acid, alkali or vegetable matter.

Coloring. 5. If artificial coloring material is required, only mineral colors shall be used.

Reinforcement. 6. The reinforcing metal shall meet the requirements of the Standard Specifications for Steel Reinforcement adopted March 16, 1910, by the American Railway Engineering Association.

Joint Filler. 7. The expansion joint filler shall be a suitable elastic waterproof compound that will not become soft and run out in hot weather, nor hard and brittle and chip out in cold weather.

SUB-GRADE.*

Sub-Grade. 8. *Slope.*—The sub-grade shall have a slope toward the curb of not less than one-half ($\frac{1}{2}$) in. per ft.

9. *Depth.**—(*a*) The sub-grade shall not be less than eleven (11) in. below the finished surface of the walk.

(*b*) The sub-grade shall not be less than five (5) in. below the finished surface of the walk.

10. *Preparation.*—All soft and spongy places shall be removed and all depressions filled with suitable material which shall be thoroughly compacted in layers not exceeding six (6) in. in thickness.

11. *Deep Fills.*—When a fill exceeding one (1) ft. in thickness is required to bring the work to grade, it shall be made in a manner satisfactory to the engineer. The top of all fills shall extend beyond the walk on each side at least one (1) ft. and the sides shall have a slope not greater than one (1) on one and one-half ($1\frac{1}{2}$).

Drainage. 12. When required, a suitable drainage system shall be installed and connected with sewers or other drains indicated by the engineer.

SUB-BASE.*

Sub-Base. 13. *Width—Thickness.*—On the sub-grade shall be spread a suitable material as hereinbefore stated which shall be thor-

* When a sub-base is required, eliminate paragraph 9 (*b*). When a sub-base is not required, eliminate paragraphs 3 and 9 (*a*). Unless paragraph 9 (*a*) is eliminated, 9 (*b*) is void

oughly rolled or tamped to a surface at least five (5) in. below the finished grade of the walk. On the fills, the sub-base shall extend the full width of the fill and the sides shall have the same slope as the sides of the fill.

14. *Wetting.*—While compacting the sub-base, the material shall be kept thoroughly wet and shall be in that condition when the concrete is deposited.

FORMS.

Forms.

15. *Materials.*—Forms shall be free from warp and of sufficient strength to resist springing out of shape.

16. *Setting.*—The forms shall be well staked or otherwise held to the established lines and grades and their upper edges shall conform to the established grade of the walk.

17. *Treatment.*—All wood forms shall be thoroughly wetted and metal forms oiled before depositing any material against them. All mortar and dirt shall be removed from forms that have been previously used.

CONSTRUCTION.

Slabs.

18. *Size of Slabs.*—The slabs or independently divided blocks when not reinforced shall have an area of not more than thirty-six (36) sq. ft. and shall not have any dimension greater than six (6) ft. Larger slabs shall be reinforced as hereinafter specified.

19. *Thickness of Walk.*—The thickness of the walks should not be less than five (5) in. for residence districts, and not less than six (6) in. for business districts.

Joints.

20. *Width and Location of Joints.*—A one-half ($\frac{1}{2}$) in. expansion joint shall be provided at least once in every fifty (50) ft.

Edges.

21. *Protection of Edges.*—Unless protected by metal, the upper edges of the concrete shall be rounded to a radius of one-half ($\frac{1}{2}$) in.

MEASURING AND MIXING.

Measuring.

22. *Measuring.*—The method of measuring the materials for the concrete, including water, shall be one which will insure separate uniform proportions at all times. A bag of Portland cement (94 lb. net) shall be considered one (1) cu. ft.

Mixing.

23. The ingredients of the concrete or mortar shall be thoroughly mixed dry, sufficient water added to obtain the desired consistency, and the mixing continued until the materials are uniformly distributed and the mass is uniform in color and homogeneous.

(*a*) *Machine Mixing.*—When the conditions will permit, a machine mixer of the type that insures the uniform proportioning of the materials throughout the mass, shall be used.

(*b*) *Hand Mixing.*—When it is necessary to mix by hand, the mixing shall be on a water-tight platform and the materials shall be turned until the mass is uniform in color and homogeneous.

Retempering.

24. *Retempering*, that is, remixing mortar or concrete that has partially hardened with additional water, will not be permitted.

TWO-COURSE WALK.

BASE.

Two-Course Walk. Base.

25. *Proportions.*—The concrete shall be mixed in the proportion by volume of one (1) part Portland cement, two and one-half ($2\frac{1}{2}$) parts fine aggregate and five (5) parts coarse aggregate.

26. *Consistency.*—The materials shall be mixed wet enough to produce a concrete of a consistency that will flush readily under slight tamping, but which can be handled without causing a separation of the coarse aggregate from the mortar.

27. *Placing.*—After mixing, the concrete shall be handled rapidly and the successive batches deposited in a continuous operation completing individual sections. Under no circumstances shall concrete be used that has partially hardened. The forms shall be filled and the concrete struck off and tamped to a surface the thickness of the wearing course below the established grade of the walk. After the concrete has been thoroughly tamped against the cross forms, they shall be removed and the material for the adjoining slab deposited so as to preserve the joint. Workmen shall not be permitted to walk on the freshly laid concrete, and if sand or dust collects on the base it shall be carefully removed before the wearing course is applied.

Reinforcement.

28. Slabs having an area of more than thirty-six (36) sq. ft., or having any dimension greater than six (6) ft., shall be reinforced with wire fabric or with plain or deformed bars. The

cross sectional area of metal shall amount to at least 0.041 sq. in. per lin. ft. The reinforcement shall be placed upon and slightly pressed into the concrete base immediately after the base is placed. Reinforcement shall not cross joints and shall be lapped sufficiently to develop the strength of the metal.

WEARING COURSE.

Wearing Course.

29. *Proportions.*—The mortar shall be mixed in the manner hereinbefore specified in the proportion by volume of one (1) part Portland cement, and not more than two (2) parts fine aggregate.

30. *Consistency.*—The mortar shall be of a consistency that will not require tamping, but which can be easily spread into position.

31. *Thickness.*—The wearing course of the walk in residence districts shall have a minimum thickness of three-quarters ($\frac{3}{4}$) in., and in business districts a minimum thickness of one (1) in.

32. *Placing.*—The wearing course shall be placed immediately after mixing and in no case shall more than fifty (50) minutes elapse between the time the concrete for the base is mixed and the time the wearing course is placed.

33. *Finishing.*—After the wearing course has been brought to the established grade, it shall be worked with a wood float in a manner that will thoroughly compact it. When required, the surface shall be troweled smooth, but excessive working with a steel trowel shall be avoided. The slab markings shall be made in the wearing course directly over the joints in the base with a tool which will completely separate the wearing course of adjacent slabs. If excessive moisture occurs on the surface, it must be taken up with a rag or mop and in no case shall dry cement, or a mixture of dry cement and sand, be used to absorb this moisture or to hasten the hardening. Unless protected by metal, the surface edges of all slabs shall be rounded to a radius of about one-half ($\frac{1}{2}$) in.

Coloring.

34. If artificial coloring is used, it must be incorporated with the entire wearing course and shall be mixed dry with the cement and aggregate until the mixture is of uniform color. In no case shall the amount of coloring used exceed five (5) per cent of the weight of the cement.

ONE-COURSE WALK.

One-Course Walk.

The general requirement of the specifications covering two-course work will apply to one-course work with the following exceptions:

35. *Proportions.*—The concrete shall be mixed in the proportion of one (1) part Portland cement to one and two (2) parts fine aggregate, and three (3) parts coarse aggregate passing a one (1) in. ring.

36. *Placing and Finishing.*—The form shall be filled, the concrete struck off and the coarse particles forced back from the surface, and the work finished in the usual way.

37. *Reinforcement.*—When a single course walk is to be reinforced, the metal shall be placed in the middle of the section. The minimum amount of metal shall be as specified in Paragraph 28.

PROTECTION.

Protection.

38. *Treatment.*—As soon as the concrete has hardened sufficiently to prevent being pitted, the surface of the walk shall be sprinkled with clean water and kept wet for at least four (4) days. The walk shall not be opened to traffic until the engineer so directs.

39. *Temperature below 35° F.*—If at any time during the progress of the work the temperature is or, in the opinion of the engineer, will within twenty-four (24) hours drop to thirty-five degrees (35°) F., the water and aggregates shall be heated and precautions taken to protect the work from freezing for at least five (5) days. In no case shall concrete be deposited upon a frozen sub-grade or sub-base.

National Association of Cement Users.

HARRISON BUILDING,
PHILADELPHIA, PA.

STANDARD SPECIFICATIONS FOR CONCRETE ROADS AND STREET PAVEMENTS.

MATERIALS.

1. The cement shall meet the requirements of the Standard Specifications for Portland Cement of the American Society for Testing Materials and adopted by this Association (Standard No. 1). **Cement.**

2. The aggregates shall be clean, coarse, hard, durable materials and shall be free from dust, soft, flat or elongated particles, loam, vegetable or other deleterious material. In no case shall aggregate containing frost or lumps of frozen material be used. **Aggregates.**

(*a*) *Fine Aggregate for Concrete.*—Fine aggregate shall consist of sand, crushed stone or gravel screenings preferably of silicious material, graded from fine to coarse and passing, when dry, a screen having one-quarter ($\frac{1}{4}$) in. diameter holes, and not more than three (3) per cent shall pass a sieve having one hundred (100) meshes per lin. in.

Fine aggregate shall be of such quality that mortar composed of one (1) part Portland cement and three (3) parts fine aggregate, by weight, when made into briquettes, will show a tensile strength at least equal to the strength of 1:3 mortar of the same consistency made with the same cement and Standard Ottawa sand.

(*b*) *Aggregate for Wearing Course.*—The aggregate shall consist of screened gravel or stone screenings from granite or other close-grained durable rock sufficiently hard to scratch glass, mixed in the proportion of three (3) parts passing a one-half ($\frac{1}{2}$) in. ring and retained on a screen having one-quarter ($\frac{1}{4}$) in. diameter holes and two (2) parts passing a screen having one-quarter ($\frac{1}{4}$) in. diameter holes and retained on a screen having fifty (50) per lin. in.

(*c*) *Coarse Aggregate for Concrete.*—Coarse aggregate shall consist of inert materials such as stone or gravel, graded in size, retained on a screen having one-quarter ($\frac{1}{4}$) in. diameter holes and the maximum size shall be such as to pass a one and one-half ($1\frac{1}{2}$) in. ring.

(*d*) *Natural Mixed Aggregates.*—Natural mixed aggregates shall not be used as they come from deposits, but shall be screened and remixed to agree with the proportions specified.

Sub-Base. 3. Only clean, hard, suitable material, not exceeding four (4) in. in the largest dimension, shall be used in the sub-base.*

Water. 4. Water shall be clean, free from oil, acid, alkali, or vegetable matter.

Color. 5. If artificial coloring matter is required, only mineral colors shall be used.

Reinforcement. 6. The reinforcing metal shall meet the requirements of the Standard Specifications for Steel Reinforcement adopted March 16, 1910, by the American Railway Engineering Association.

Joint Filler. 7. The expansion joint filler for open joints shall be a suitable bitumen that will not become soft in hot weather, nor hard and brittle in cold weather. Expansion joints may also be formed by inserting during construction, and leaving in place, a total thickness of one-quarter ($\frac{1}{4}$) in. of tarred paper or tarred felt.

SUB-GRADE.*

Sub-Grade. 8. *Section.*—The sub-grade shall have a rise at the center of not more than one-hundredth (1/100) the width of the pavement.

9. *Depth.**—(*a*) The sub-grade shall not be less than twelve (12) in. below the finished surface of the pavement.

(*b*) The sub-grade shall not be less than six (6) in. below the finished surface of the pavement.

10. *Preparation.*—All soft and spongy places shall be removed and all depressions filled with suitable material which shall be thoroughly compacted in layers not exceeding six (6) in. in thickness.

11. *Deep Fills.*—When a fill exceeding one (1) ft. in thick-

* When a sub-base is required, eliminate paragraph 9 (*b*). When a sub-base is not required, eliminate paragraphs 3, 9 (*a*), 13 and 14. Unless paragraph 9 (*a*) is eliminated, 9 (*b*) is void.

ness is required to bring the pavement to grade, it shall be made in a manner satisfactory to the engineer.

12. *Drainage.*—When required, a suitable drainage system shall be installed and connected with sewers or other drains indicated by the engineer. Drainage.

SUB-BASE.

13. *Thickness.*—On the sub-grade shall be spread a material as hereinbefore specified, which shall be thoroughly rolled and tamped to a surface at least six (6) in. below the finished grade of the pavement. Sub-Base.

14. *Wetting.*—While compacting the sub-base, the material shall be kept thoroughly wet and shall be in that condition when the concrete is deposited.

FORMS.

15. *Materials.*—Forms shall be free from warp and of sufficient strength to resist springing out of shape. Forms.

16. *Setting.*—The forms shall be well staked or otherwise held to the established lines and grades and their upper edges shall conform to the established grade of the pavement.

17. *Treatment.*—All wood forms shall be thoroughly wetted and metal forms oiled before depositing any material against them. All mortar and dirt shall be removed from forms that have been previously used.

EXPANSION JOINTS.

18. *Width and Location.*—Expansion joints not less than one-quarter ($\frac{1}{4}$) in. nor more than one-half ($\frac{1}{2}$) in. in width shall be placed across the street or road not more than twenty-five (25) ft. apart, perpendicular to the center line. When a curb or combination curb and gutter is used, a one-half ($\frac{1}{2}$) in. joint shall be placed between it and the pavement. All expansion joints shall extend through the entire thickness of the pavement. Expansion Joints.

19. *Protection of Edges.*—When required by the engineer in charge, the concrete at the expansion joints shall be protected with metal. Unless protected by metal or filled with tarred paper or felt, the upper edges of the concrete shall be rounded to a radius of three-eighths ($\frac{3}{8}$) in.

MEASURING AND MIXING.

Measuring.

20. *Measuring.*—The method of measuring the materials for the concrete, including water, shall be one which will insure separate, uniform proportions at all times. A bag of Portland cement (94 lb. net) shall be considered one (1) cu. ft.

Mixing.

21. The ingredients of the concrete or mortar shall be thoroughly mixed dry, sufficient water added to obtain the desired consistency, and the mixing continued until the materials are uniformly distributed and the mass is uniform in color and homogeneous.

(*a*) *Machine Mixing.*—When the conditions will permit, a machine mixer of a type which insures the uniform proportioning of the materials throughout the mass, shall be used.

(*b*) *Hand Mixing.*—When it is necessary to mix by hand, the mixing shall be on a water-tight platform and the materials shall be turned until the mass is uniform in color and homogeneous.

Retempering.

22. *Retempering*, that is, remixing mortar or concrete that has partially hardened with additional water, will not be permitted.

TWO-COURSE PAVEMENT.

BASE.

Two-Course Work. Base.

23. *Proportions.*—The concrete shall be mixed in the proportion by volume of one (1) part Portland cement, two and one-half ($2\frac{1}{2}$) parts fine aggregate and five (5) parts coarse aggregate.

24. *Consistency.*—The materials shall be mixed with sufficient water to produce a concrete of a consistency such that mortar will flush to the surface under light tamping, but which can be handled without causing a separation of the coarse aggregate from the mortar.

25. *Placing.*—After mixing, the concrete shall be handled rapidly into place and successive batches deposited in a continuous operation, completing sections between expansion joints without the use of intermediate cross forms or bulkheads. Concrete that has partially hardened shall not be used. The concrete shall be well tamped to a surface the thickness of the wearing course below the established grade of the pavement. Workmen shall not walk on the freshly laid concrete and if sand or dust collects on the base, it shall be carefully removed before the wearing course is applied.

Reinforcement.

26. On streets more than twenty (20) ft. wide not having car tracks, the pavement shall be reinforced with wire fabric or with plain or deformed bars. The cross sectional area of metal shall amount to at least 0.041 sq. in. per ft. measured parallel to the axis of the street and at least 0.025 sq. in. per ft. measured perpendicular to the axis of the street. The reinforcement shall be placed upon and slightly pressed into the concrete base immediately after the base is placed. Reinforcement shall not cross expansion joints and shall be lapped sufficiently to develop the strength of the metal.

WEARING COURSE.

Wearing Course.

27. *Proportions.*—The mortar shall be mixed in the manner hereinbefore specified in the proportion by volume of one (1) part Portland cement and not more than two (2) parts aggregate for wearing course.

28. *Consistency.*—The mortar shall be of a consistency that will not require tamping but which can be easily spread into position with a template or straight-edge.

29. *Thickness.*—The wearing course of the pavement in residence districts shall have a minimum thickness of one and one-half ($1\frac{1}{2}$) in. and in business districts a minimum of two (2) in.

30. *Placing.*—The wearing course shall be placed immediately after mixing, and in no case shall more than fifty (50) minutes elapse between the time the concrete for the base is mixed and the time the wearing course is placed.

31. *Finishing.*—After the wearing course has been brought to the established grade with a template or straight-edge, it shall be worked with a wood float in a manner that will thoroughly compact it and produce a comparatively smooth surface, free from depressions or irregularities of any kind. The finished surface of the concrete shall not vary more than one-quarter ($\frac{1}{4}$) in. from a two (2) ft. straight-edge placed upon the concrete in any position.

Coloring.

32. If artificial coloring is used, it must be incorporated with the entire wearing course and shall be mixed dry with the cement and aggregate until the mixture is of uniform color. In no case shall the amount of coloring used exceed five (5) per cent of the weight of the cement.

ONE-COURSE PAVEMENT.

One-Course Work.

The general requirements of the specifications covering two-course work will apply to one-course work with the following exceptions:

33. *Proportions.*—The concrete shall be mixed in the proportion by volume of one (1) part Portland cement to two (2) parts fine aggregate (Paragraph 2 *a*) or aggregate for wearing course (Paragraph 2 *b*) and three (3) parts coarse aggregate passing a one (1) in. ring.

34. *Placing and Finishing.*—The concrete shall be placed as provided for under Two-Course Pavement, base and wearing course respectively.

35. *Reinforcement.*—When a one-course pavement is to be reinforced, the metal shall be placed at the middle of the section. The minimum amount of metal shall be as specified under Two-Course Pavement.

CROWN.

Crown.

37. *Amount.*—All types of concrete pavement shall be given a rise or crown at the center of at least one one-hundredth (1/100) but not more than one seventy-fifth (1/75) of the width of the pavement. A portion of this crown may be obtained by increasing the thickness of the pavement at the center rather than by laying a pavement of uniform thickness on a crowned sub-grade or sub-base.

PROTECTION.

Protection.

37. *Treatment.*—As soon as the concrete has hardened sufficiently to prevent being pitted, the surface of the pavement shall be sprinkled with clean water and shall be kept wet for at least four (4) days. Concrete pavement on roads shall be covered as soon after finishing as it is possible to do so without damaging the surface, with at least two (2) in. of dirt, which shall be kept wet for at least four (4) days. Before covering with dirt the pavement shall be sprinkled with water as above specified. The pavement shall not be open to traffic until the engineer so directs.

SHOULDERS.

39. *Construction.*—On streets where the pavement does not occupy the full width of the street, and on roads, a gravel or crushed stone shoulder at least two (2) ft. wide shall be constructed on each side of the pavement. The surface of the shoulders shall have a slope away from the pavement of one and one-half ($1\frac{1}{2}$) in. per ft. and a thickness for the two (2) foot width adjoining the concrete at least equal to the minimum thickness of the concrete.

WEARING SURFACE OF BITUMEN AND SAND.

Wearing Surface of Bitumen and Sand.

40. *Construction.*—Where a wearing surface of bitumen and fine aggregate is used it shall preferably be placed upon a one-course pavement constructed as hereinbefore specified, but may be used also on two-course work.

41. *Expansion Joints.*—Before applying the bitumen to the concrete, all open expansion joints in the pavement shall be filled as hereinbefore specified. Where required by the engineer in charge, the concrete at the expansion joints shall be protected with metal.

42. *Bitumen.*—The bitumen shall be of a quality specified by the engineer.

43. *Placing.*—After the concrete base has hardened for at least seven (7) days, the thoroughly clean, dry surface of the pavement shall be covered with hot bitumen applied with a sprinkling wagon designed for the purpose, or with suitable hand sprinkling cans. The hot bitumen shall be evenly distributed over the concrete by brushing with suitable brooms and then covered with the fine aggregate. (Paragraph 2 *b*.)

44. *Amount of Bitumen and Sand.*—Approximately one-half ($\frac{1}{2}$) gallon of bitumen shall be applied per square yard of pavement. Approximately one (1) cu. yd. of fine aggregate shall be applied per one hundred and fifty (150) sq. yd. of pavement.

45. *Protection.*—The pavement shall not be opened to traffic until the engineer so directs.

THE CONCRETE ROADS OF WAYNE COUNTY, MICHIGAN.

By Edward N. Hines.*

It is a sad commentary on the conduct of an undertaking of any magnitude, that individuals, municipalities, states or nations all seem to find it necessary to do a certain amount of experimenting and dilly-dallying before accepting the conclusions and avoiding the failures of previous demonstrations. This is particularly true of the various phases of the good roads' movement.

With four years' experience as a guide, it has been demonstrated in Wayne County that a well-built concrete road is a practical form of construction which merits and will receive a more extensive adoption. Every test to which this work has been subjected only emphasizes its strong characteristics. The points considered, and which might properly be termed a specification, are initial cost, ultimate cost, which includes maintenance, sanitation and freedom from dust, good traction for all types of vehicles, smoothness and ease of construction.

The initial cost of a good concrete road is little, if any, greater than that of a first-class bituminous macadam road. One of the greatest fallacies indulged in by communities starting to improve their highways, is that cheapness in cost of original construction of roads means economy and that the highway official who can build the greatest area of roads at the least outlay per square yard, is working for the communities best interest.

The first two miles of Grand River road, one of the main highways out of Detroit, is tar macadam. If someone had offered to build this road absolutely without one penny's cost to Wayne County, stipulating only that it should maintain it in a fairly average condition, at the end of eight years it would have been money ahead by rejecting the offer and building it of concrete under its present specifications. Of course, six years ago, when this road was built, it did not possess this knowledge and the experience since acquired was one of the reasons for abandoning

* Member Board of Wayne County Road Commissioners, Detroit, Mich.

the construction of this type of road and turning to concrete. When it comes to annual cost, the concrete road stands pre-eminent. With over sixty miles of concrete road in Wayne County, some of it in its fourth year, less than $300 has been spent for maintenance, and this is what makes this type of road the cheapest of all good roads.

The Woodward Avenue road, now in its fourth year, shows little or no signs of wear, although it is not built nearly so well as our latest constructed concrete roads. A conservative estimate of traffic on this road shows that over 1,300,000 vehicles (more traffic than would go over an ordinary country road in twenty years) have passed a given point without the development of ruts, holes or bumps and with the expenditure of next to nothing for surface maintenance. On this same road, Woodward Avenue, at the Eight Mile road, in Oakland County, adjoining the Wayne County concrete road, Royal Oak Township built this year two miles of asphalt macadam at a cost well over $1.00 per sq. yd.; the first mile was opened for traffic August 17th. On November 30th the speaker went over this mile and counted 137 holes from one foot up to four, five and six feet square. Fifteen men, a steam roller and a couple of teams were at work patching and repairing this road, not four months old. If this record is not convincing as to the merits of concrete construction for roads, a test worked out recently in Detroit adds further testimony.

About a year ago, as a result of the rapid deterioration of new pavements, the City of Detroit conducted an inquiry into materials used and methods followed in paving, and the common council of the city voted $600 for testing purposes to decide the merit, or lack of merit, of the various forms of built-up pavements. Boiler Inspector McCabe designed a very ingenious machine whereby all the conditions met with in street traffic could be reproduced as nearly as possible.

The machine is made to revolve about a vertical shaft which carries on a bearing, a head, which forms a bearing for the steel frame, at each end of which are jointed T-castings. These T-castings carry the shafts upon which solid cast-iron wheels are mounted. The wheels have a 3-in. tire, carry each a load of 1,350 lb., and are not supplied with springs or shock absorbers of any description. Each shaft also carries several plungers on which

are mounted horse shoes, each one of which strikes a 75-lb. blow. The wheels are moved across the pavement by a train of gears. The dimension from outside to outside is 19 ft. 8 in., the greatest wheel radius is 8 ft. 7 in. It requires 333 revolutions about the circle to move the wheels across the pavement 2 ft., which is the limit of travel. The foundation of the pavement consists of 8 in. of concrete. The paving is laid on 2 in. of sand, which is packed by a hand pounder until firm and uniform. The concrete sample is 6 thick and laid on the foundation. In the first experiment, sections of cedar block, granite block, creosote block and three or four varieties of brick and concrete were tried out. The Detroit *Free Press* reports as follows:

"In the first experiment with the machine, it was run at a comparatively high speed to give it a severe test, and the pavement which stood up best under this punishment was the concrete laid under the specifications of the Wayne County Road Commission. The wear on the surface was hardly perceptible, while the same wear on granite block and all brands of brick tried was as great as 1½ in."

Whenever road builders, highway commissioners or others interested in highway construction get together they talk maintenance, maintenance and more maintenance. While not belittling the principle of maintaining a road after it is built and following it out in practice, it seems to the speaker, with Wayne County's experience, it would pay other communities to adopt a form of construction on which it is not necessary to expend from $800 to $1,300 a mile yearly to keep it in fairly usable condition. The Wayne County concrete roads are sanitary, as there is no detritus from the road itself; there are few cracks and joints to hold dirt and animal droppings and there is no dust. The drier the weather, the less dirt on them, as vehicles do not track mud from unimproved cross roads in dry weather. What little dirt is tracked on it is immediately blown off or washed off by the first rain. These concrete roads have a gritty surface and are not slippery in any kind of weather, affording good traction for all types of vehicles. Horses find good footing on them and automobiles do not skid in wet weather. It is not necessary to build concrete roads with any great amount of crown, and the tendency to drive in tracks, resulting in the formation of ruts, so apparent in macadam roads, is eliminated, as the driver of a vehicle can sit comfortably in his

seat, no matter on what part of the road he may be driving; neither can a horse pick out the beaten track as on a gravel or macadam road, but must be driven or he will zig-zag over the entire road. A crown of $\frac{1}{4}$ in. to the foot disposes of the surface water and tends to distribute traffic over the entire area of the road. Another desirable feature of concrete roads is smoothness. The speaker quotes from a report of the Committee on Pavements, appointed in October, 1911, by Mayor Gaynor of New York City, to investigate and report to him on the present conditions of its pavements:

"No quality which a pavement can possess is more important than smoothness. Every irregularity in the surface is a source of weakness and of ultimate failure. As the wheels are drawn over the road, the wear which they cause is almost in proportion to the obstacles encountered. If the pavement is rough, as our stone ones are, or if it be broken, the wheels pound, and the pavement is subjected to heavy blows which soon wear it away and otherwise destroy it. In almost all our stone pavements, one can find places where the blocks have actually been crushed or split from this cause. If the pavement is of wood block, asphalt or any other composition material, and the surface is wavy, the depressions will hold water and speedily lead to failure; in such pavements disintegration almost invariably commences in these places. In macadam roads, depressions of this sort are the chief cause of wear, and especially so since the introduction of the automobile. The rapidly passing wheels throw out the standing water with great violence, carrying with it the binder or fine stuff between the stones, thus causing the pot holes which make their appearance so rapidly on such roads when subjected to heavy automobile traffic.

"It should be remembered that since the advent of the automobile, smoothness for pavements is an even more important quality than it was formerly. The shock which a swiftly moving vehicle receives when it meets an obstruction, is more violent and destructive in its effects than if it were proceeding at a more moderate gait, and no matter how the force of the blow may be disguised from those riding in the car, by springs, pneumatic tubes or otherwise, the destructive effect of the blow remains the same, and is absorbed by some part of the mechanism, causing injury either to the tires or frame.

"The loss sustained yearly by the citizens who use automobiles by reason of the roughness of our pavements, must be very great in the aggregate."

With all the other good points in its favor, concrete can be handled with comparative ease, and providing the work is carried on under skilled supervision, can be laid with working force of relatively unskilled labor. It must be borne in mind, however, that the addition of a little cement to a quantity of stone and sand does not make concrete. There is no material which will respond so quickly to a little care, and if proper attention is given

to the detail of mixing and curing, it will repay you in quality and permanence. With the foregoing in mind as to why Wayne County is continuing to build concrete roads, the speaker will take a little more time and tell how it is building them.

Drainage and a good foundation are necessary for any type of road, and for a concrete road, the greater care that is taken in this respect, the better will be the final result. A well-drained, well-compacted sub-grade will eliminate cracks to a very large degree. Drainage areas do not restrict themselves to township boundaries nor to legislative edicts concerning jurisdiction, and owing to the fact that the commission's jurisdiction embraces only Wayne County and at the present time only about 120 miles out of a total 1,370 miles of road therein, it has been handicapped to a considerable extent in both these problems. The county is flat, and although some few sections are sandy, the sub-soil is largely of a heavy, sticky clay, therefore not easily drained. As at the present time the commission has neither the money nor jurisdiction to reconstruct the entire system of drainage and ditches throughout Wayne and the adjacent counties, it does the best it can under the circumstances.

Relative to foundations, the roads had to be built and widened on fills; a year's time to settle would have vastly improved conditions, but the immediate demand for the road was too insistent to permit delay.

One of the alleged bad features of concrete roads is the tendency to crack. In order to overcome this tendency, the sub-grade is prepared as carefully as conditions permit, making it flat and rolling it hard and firm. Due to temperature changes and the absorption of water, concrete is constantly in motion, and the flat sub-grade tends to overcome frictional resistance and thereby prevents longitudinal cracking. In the first concrete road built, the sub-grade was conformed to the finished crown of the road and there was used what the speaker terms, for the want of a better name, an inverted curb. In this road, and on the first concrete road built on Michigan Avenue, where practically the whole road is built on a fill, more cracks have developed than on all subsequent construction. These cracks, however, are well taken care of at small expense, by the use of hot refined tar and sand. It is the repair of these cracks that makes the surface maintenance

cost, and with a well-drained, well-rolled, firm, sub-grade, cracks of all kind are reduced to a minimum and need not be seriously considered. The roads are built in 25-ft. sections to provide for contraction and expansion, believing it wise to form lateral cracks beforehand so that the edges can be properly protected from chipping and spalling, using a metal plate which is a development of previous experiments. This plate is about $\frac{3}{16}$ in. thick and 3 in. wide, provided with shear members which anchor it securely to the concrete base and wearing surface. It is shaped to conform to the crown of the finished road, and two thicknesses of three-ply asphalted felt (about $\frac{1}{4}$ in.) are inserted between the two plates of each joint. By the use of these plates the wear at the joint, which is the weakest point in a concrete road, has been practically overcome, besides securing a smooth, even, continuous surface.

Wayne County is poor in good road material and everything has to be imported. The best results have been secured from the use of washed gravel ranging in size from $\frac{1}{4}$ in. to $1\frac{1}{2}$ in., and washed sand from $\frac{1}{8}$ in. to nothing. This material is well graded so as to secure a dense aggregate. Freedom from loam, clay or other foreign matter is absolutely insisted upon. A rich mix is required, using one part Portland cement to three parts of stone, with just a little more than enough sand to fill the voids in the stone.

The detail of mixing and curing the concrete has been as great a factor as any other in the success of these roads. The roads have a minimum thickness of 7 in.; after the sub-grade is prepared, side rails are placed of 2 x 7 in. lumber protected on top by a 2-in. angle iron; the concrete, thoroughly mixed to a wet consistency, is laid on the sub-grade, which is well sprinkled just previous to placing the concrete, to prevent the water in the concrete being absorbed by capillary attraction; no tamping is necessary, although a couple of men are required to work it with shovels; it is neither wise nor desirable to have the mortar and fine aggregate worked to the top, as it is the stone which should receive the wear. After the concrete is in place, no workman is permitted in any manner to disturb the finished surface by stepping or throwing anything on it. Two men move backward and forward over the surface of the concrete a plank, formed to the curvature of the road, iron bound on the edges and resting on the side rails or form boards at

the sides of the concrete over which this strike-off rides smoothly. It is handled with sufficient care to eliminate the necessity for any considerable floating by the follow-up men. These follow-up men or floaters work on a bridge which rests on the form boards or rails at the side of the road, so there is never any contact with the concrete. The final "smoothing up" is done with wooden floats of home manufacture. When the concrete has become sufficiently firm to prevent the removal of the side rails, the finishers, to prevent a sharp division line between the concrete and the gravel shoulders, pare the outer edges which are formed next to the rails.

Each day's work is finished up to an expansion joint, and no more than twenty minutes is permitted to elapse between batches during the day. The work of the day is covered with canvas, and the next day the canvas is removed; to prevent the concrete from drying out too rapidly, it is covered to the depth of about 2 in. with any sand or loose soil that may be available. The concrete is sprinkled continuously for eight days and the roads are not opened for traffic until at least two weeks after the last concrete is placed.

Trunk roads are built 16 ft. and secondary roads 15 ft. wide, with a minimum of 24 ft. over all. Concrete roads have also been built with the metal 18, 12 and 10 ft. wide. The shoulders are usually built of limestone or gravel in two layers of 3 in. each and rolled with a ten-ton roller. This work is not started until the adjacent concrete is at least three weeks old.

Wayne County does all work under the day-labor plan, and during the busy season employs as many as 1,200 workmen; handles 900 to 1,000 cars of material a month and builds about a mile of road every three days. Machinery plays an important part; it is believed that man should not be set at a task which a machine can do as well or better. Stone, sand and cement are hauled from railroad sidings to the job by steam traction engines or combination traction engines and rollers; graders, rooter plows and scarifiers are hauled in the same manner, doing the work of from six to eight horses more efficiently and more rapidly. Each concrete gang uses about 15,000 gallons of water a day, which is pumped from the nearest available source, through a 2-in. pipe, with a tap every 400 feet, laid along the road under improvement;

gasoline engines are used and water has been pumped six miles. Where there is room along the railroad side tracks, a hoist with a ton bucket for transferring stone and sand to the hauling wagons, is used. Concrete is mixed in a mechanical batch mixer that travels under its own power, and from which a boom projects, capable of being swung in the arc of a semi-circle. The men are housed and fed in camps along the roadside. Many other small economies, improvements and labor-saving devices have been worked out which tend to keep the quality up and the cost down. All work is specialized; one crew prepares the grade; another gets the material on the grade, which is done with such nicety as to make it usually unnecessary to haul in extra sand or pebbles to make a properly proportioned batch; another crew handles the concrete; another builds the shoulders, etc. By so doing, the men become more efficient as they become familiar with their tasks and more and better work is secured.

System is the key-note of the whole organization, and when one considers that the commission is conducting a business enterprise, spending $600,000 a year, employing from 1,000 to 1,200 men, working simultaneously at points 40 miles apart; that it is handling around 1,000 cars of material a month, receiving daily reports, making up payrolls, tracing shipments, keeping crews supplied with materials, and handling a voluminous correspondence; that all these and the other details of a big business are handled in the office by four people, at an expense to the county of $6,225 a year, some idea may be had of the methods employed. While there has been no intention or desire to be extravagant or wasteful, yet the Commission has done and will continue to do all things which in its judgment seem desirable to give the people of Wayne County a system of good roads, bridges and culverts, at cost; believing annual cost, rather than first cost, and permanency and utility, rather than cheapness, to be the ends sought.

DISCUSSION.

Mr. Maynard. MR. ARTHUR J. MAYNARD.—What was the cost of the roads; what mixture of concrete did you use?

Mr. Hines. MR. EDWARD N. HINES.—The proportion used was 1 : $1\frac{1}{2}$: 3. The cost was as low as $1.04 and as high as $1.75, depending on the length of haul, cost of materials at the sidings, the drainage, etc. Everything is included in the cost; shoulders, ditching, engineering expense, etc., and figured wholly on the yardage of the concrete. For the past two years we have been building shoulders of limestone or gravel. We use gravel in our concrete at the present time, one mixture; the aggregate varying in size from $1\frac{1}{2}$ in. to $\frac{1}{4}$ in. well graded, screened and washed. Our idea is to have the coarse aggregate take the wear rather than the cement and sand. We use screened and washed sand from $\frac{1}{8}$ in. down to dust, with the coarser particles predominating. Our concrete roads are crowned $\frac{1}{4}$ in. to the foot and laid in 25-ft. sections. Expansion and contraction is taken care of by a joint, cut entirely through the concrete, with an asphalted felt filler and protected at the edges with steel plates that remain permanently. When we first started using these steel plates, we had a form board to separate the plates and when we drew it out we found it had a tendency to move the concrete and form a ridge or depression. With the use of the steel plate and asphalt felt or tarred paper that is left in permanently, we obtain a smooth, even surface, which prevents the concrete spalling off or chipping at the edges.

The first road that we built developed quite a few longitudinal cracks and we have taken care of these by filling them with refined tar, heated to about 225° F., and covering with coarse dry sand that the traffic works into them. This accounts for the low maintenance cost on our sixty-odd miles of concrete; this was done last year on the first work we laid and there has been no other maintenance charge. These cracks were due largely to our lack of experience. We believe that it pays us to take a great deal of care with the sub-grade and the same care in finishing. We do not let traffic on these roads until at least two weeks after

the concrete has been placed, at which time the shoulders are placed; in cold weather the time is very much longer. We have closed some roads six weeks through very cold weather. We have not found that frost has any effect on these roads. I do not believe it is necessary to put cinders or any other material on the natural sub-soil if you make proper provision for expansion. That is only an opinion based on experience. Mr. Hines.

On some forty or fifty miles, so few cracks have developed that they are practically negligible. The first year's work on Michigan Avenue, where the road was widened from 14 to 24 ft., and one-half was built on the old roadway and the other half on a fill, a good many longitudinal cracks developed; but it is easy to understand why, as some of that fill was built only ten days or two weeks before the concrete was placed.

Our laws are a little different from other state road laws. The unit is the county, but before the provisions of the county road system can become effective it is necessary that a referendum be taken in that particular county. This law has been on the statute books since 1893 and 43 out of 83 counties have adopted it. Wayne, the twenty-seventh county, adopted the county road system by a vote of approximately 40,000 to 7,000. Before a rod of road was built the county road commission was compelled to fight two injunction suits; a *quo warranto* proceedings, and upon the refusal of an audit on the first bill which it contracted, it started mandamus proceedings, which was decided against it in the circuit court and the case was carried to the supreme court. The decision of the supreme court compelled an appeal to the legislature to amend the law; after which it was necessary to call a special meeting of the board of supervisors, the legislative body of the county, to appoint a new board of road commissioners. Payment was refused the first debt that the new board contracted, by the board of county auditors and another mandamus proceedings was instituted. At this time the various warring interests got together and recommended that the tax which had been levied for county road purposes be paid back and a new tax levied. This necessitated another appeal to the legislature to provide an enabling act by which to legally repay the first road tax. The first tax levied was $\frac{1}{4}$ of a mill, which amounted to 23 cents on the equalized assessed valua-

Mr. Hines. tion of the county. The next county road tax was $\frac{1}{3}$ of a mill and by the time the proceeds of the first tax were spent it was found that the bituminous type of road adopted was costing too much to maintain. It was then decided to use concrete, which decision was based on the satisfaction that concrete was giving on bridge floors and street crossings under a heavy teaming. Some of these cross-walks had been down fifteen or sixteen years and were considerably higher than the grade of the adjoining road and received the severe impact of every passing vehicle. It was also felt that should the concrete not prove entirely satisfactory it would provide a good foundation for some other kind of surfacing. After a few miles of concrete road had been built the people were so well satisfied that they proposed to issue $2,000,000 worth of bonds to be expended in one year. The county road commission had to fight this proposal because it had neither sufficient experience nor organization to warrant the expenditure of such a vast sum of money. A compromise was finally effected by which the $2,000,000 was available over a period of five years. I am of the opinion that when the people want to give you more money than you can economically spend, it is a good indication of how they feel about the type of road you are building. During the coming year, provided sufficient materials to carry on our work can be obtained, some forty miles of this type of road will be built. The old saying that "A prophet is not without honor save in his own country," does not apply to Wayne County. The village of Highland Park (adjoining the first concrete road built) constructed one street of concrete three years ago; three additional streets two years ago, and last year four streets. Oakland County has also built some concrete roads, as well as the village of Hamtramck and the Canadian cities of Windsor and Walkerville, across the river. From my experience, I am convinced that the other types of roads in use elsewhere are not giving satisfaction, and I base this assertion upon the fact that during the past year over 1,000 engineers, mayors, members of boards of public works, highway commissioners and others have inspected the Wayne County work. There are many others who have come here on the same errand that we did not see. The various provinces of Canada and also the United States Office of Public Roads sent engineers to investigate these roads. If the other types of roads in use were giving

satisfaction, I do not believe Wayne County would have been visited by the number and caliber of the gentlemen who have inspected its roads. Mr. Hines.

MR. L. R. FERGUSON.—The objection against concrete roads, by those particularly who have never used them, is that either the road is too slippery or else it is too hard on the horses' feet. I would like Mr. Hines to state the result of his investigation in that line. Mr. Ferguson.

MR. HINES.—These roads are not slippery. When automobiles will not skid on them in wet weather, a team will not slip, and while they appear smooth to the eye, they have a rough, gritty surface. Of course, a sleet storm that would freeze over any pavement will freeze over a concrete road. However, there appears to be some latent heat in the concrete road so that snow and ice disappear from it quicker than they will on other types of pavement. The best testimonial as to the character of the road is from the fellow who uses it. I have at all seasons of the year stopped the farmers with the idea of getting their opinion. Most of them say, "Yes, we wear out a good many horseshoes, but we do not wear out the horses so fast, and we can cart four times the load." Mr. Hines.

MR. W. W. SCHOULER.—How soon do you cover the concrete and how long is it before the road can be used? Mr. Schouler.

MR. HINES.—We keep the road covered from the beginning for at least two weeks and sprinkle it continuously for eight days. We close the street for two weeks. We are handicapped in providing detours because most of the main highways have street car tracks, which cover the greater portion of the road. We try to make provision to go around wherever we can, through the fields or otherwise; but where that is not possible we close the road and station watchmen to furnish the traveler information as to the shortest way around. Mr. Hines.

A MEMBER.—Do you place the concrete at one operation and is it thicker in the center than at the sides? A Member.

MR. HINES.—The concrete is thicker in the center than at the sides and is all placed in one operation; it is seven inches thick at the sides. Our roads are crowned a quarter of an inch to the foot on a flat sub-grade, and the thickness in the center depends on the width of the road. Mr. Hines.

Mr. Mensch. MR. L. J. MENSCH.—Could you not make it a little thinner?

Mr. Hines. MR. HINES.—I presume so, but we think it is good practice to err on the side of safety. The committee is building roads leading into a city of 600,000 people, designed to carry a heavy mixed traffic; as these roads are continuations of the paved streets leading to the retail and wholesale districts of the City of Detroit, and as it was departing from all precedents in this work, it did not care to take any chances. It took a little bit of nerve to spend $500,000 or $600,000 on something that had never been done that way before, and we tried to be as thorough as we could, so as to avoid possible failure. Our first work is not as good as our present work. I would not want to insult your intelligence by saying that we have not made any mistakes. We have made lots of them, but we have not made the same ones twice.

Mr. Larned. MR. E. S. LARNED.—Mr. Hines do you resort to drainage on both sides of the road?

Mr. Hines. MR. HINES.—Where that is available we do. We use an open ditch on both sides of the road. Where street car tracks usurp a portion of the road we put in 3-in. land tile along the street car track and an open ditch on the other side.

Mr. Larned. MR. LARNED.—I think it is worthy of accentuation and to recognize the fact that in this work they are using a rich mixture of $1 : 1\frac{1}{2} : 3$ and still find the work economical. If the good results are attributable to the intelligent care with which the work is done, it must not be lost sight of that the rich mixture contributes to that good result, and if they used concrete of such proportions as $1 : 3\frac{1}{2} : 6$ or $1 : 4 : 8$, they would not have the same results.

Mr. Ferguson. MR. FERGUSON.—It seems to me that the key-note that they have struck in the construction of concrete roads in Wayne County is the low cost of maintenance. If it costs 5 cents per sq. yd. more to maintain the usual type of road than one of concrete, it will pay to make an investment of a dollar more a year in a concrete road than some other type of road. A road that requires 5 cents per sq. yd. for maintenance does not have anything like the length of life that the concrete road has.

Another point Mr. Hines brought up is that of joints. It occurs to me that these are expansion rather than contraction joints, and I do not believe there has been any record of buckling

effects in concrete roads. The concrete in setting shrinks to a considerable extent, probably as much as would be caused by a change in temperature of 100 degrees; and when the concrete does expand it has to overcome this initial contraction before any compressive stress can develop in the concrete itself, which has sufficient strength to resist such stresses. Mr. Ferguson.

VIBROLITHIC CONCRETE PAVEMENTS.

By R. C. Stubbs.*

The vibrolithic pavement is being laid in Texas about as follows:

The sub-grade is prepared as for any other pavement; the aggregate is stored on the sub-grade. A traction concrete mixer is used having a charging bucket and a chute that delivers the concrete in the street.

The concrete is composed of one part Portland cement and five parts aggregate (sharp sand and pebbles) of which not less than 33 per cent nor more than 40 per cent shall pass a $\frac{1}{4}$-in. screen; as much water is used in mixing as the mass will retain without draining after depositing. The concrete comes direct from the chute to the street and is immediately raked to approximate grade. After a few feet has been laid the surface is float finished by means of long-handled floats. Immediately upon this surface is thrown a coating of crushed granite, graded from $\frac{5}{8}$ in. to $1\frac{1}{4}$ in.; this is applied until the surplus matrix has been taken up. (Fig. 1.)

This pavement is now ready for vibration, which is done in the following manner: Platforms are placed along or across the street, each platform is 20 in. wide made of $\frac{5}{8}$-in. x 4-in. strips cleated $\frac{1}{4}$ in. apart, and as long as desired; the cleats projecting 1 in. along one side, the adjacent platform cleat projection lapping back on to the platform. When two or three platforms have been placed, a vibrator is rolled on the first platform and passes along from one to the other as they are vibrated to grade; the concrete is thus brought to a very high density (Fig. 2). As the work progresses, the platforms are brought forward and the granite surface street is immediately covered with sharp sand and wetted.

This operation completes a pavement 6 in. thick, of 1 to 5 concrete, with a 1-in. to $1\frac{1}{2}$-in. top loaded with stone of high abrasive resistance and with unequaled load-carrying capacity. The sur-

*Contracting Engineer, Dallas, Texas.

face being of granite all parts have an equal resistance to abrasion. This granite, forced into the concrete with vibration, cannot be displaced and must wear out.

Vibrators are mechanical and can be relied upon to treat all parts alike; the operator's duty being simply to roll them over the platforms to insure a concrete of equal and even texture. Evenness of texture gives uniform resistance to internal stresses. High density means greater strength.

Two or three days later, the sand is swept off the surface into the gutter with stiff, wire brooms, and $\frac{1}{4}$-in. layer of pitch is poured

FIG. 1.—SHOWING SANDED SURFACE IN THE IMMEDIATE FOREGROUND; FINISHING PAVEMENT JUST BACK OF PLATFORMS; VIBRATOR AT WORK; CHUTE DELIVERING CONCRETE FROM PLANT AHEAD.

at a temperature of 150° F., and leveled with rubber rakes. Immediately after leveling a coating of hard stone $\frac{1}{4}$ in. to $\frac{3}{8}$ in. is spread in sufficient quantity to thoroughly cover the pitch. Upon this is spread the sand that was swept into the gutter. The surface is then rolled with a 500-lb. hand roller, and when the concrete is 7 days old the pavement is opened to traffic.

The object in covering the pavement so soon is to retain the moisture in the concrete in order that the process of hardening may be uninterrupted. The pitch may not adhere well to a moist mortar, but it takes a firm hold on the granite surface. After

30 days' traffic the surface irons down smooth, the small stone protect the pitch while the sand fills the interstices, producing a very tough surface with a thickness of about $\frac{1}{2}$ in.

The speaker does not use bituminous joints to take care of expansion and contraction; he finds that expansion results from absorption of moisture and rise of temperature, and contraction from shrinkage during the first period of hardening. Contraction will produce small cracks at irregular intervals following lines of weakness. To force them to occur in straight lines, thin wooden

FIG. 2.—SHOWING CRUSHED GRANITE SURFACE BEING SPREAD ON THE CONCRETE AND VIBRATOR AT WORK ON PLATFORMS.

planes $\frac{1}{2}$ in. x 3 in. are placed on the edge upon the prepared subgrade in such a manner that they will be buried in the concrete in the bottom half of the pavement. Within 48 hours after the concrete has been placed, thin straight lines of relief will appear over the breaker strips. In this way the street is laid in monolithic slabs; no slab being of greater size than one with a strength that will properly resist the stresses; the breakers being so distributed that no relief line is greater than $\frac{1}{16}$ in. which is barely noticeable. The speaker prefers this kind of joint because it does not wear off at the edges like a filled joint and is more economical than an armored joint.

From this description of vibrolithic pavements it can be seen that the speaker's theory of permanent pavement as an investment is to get the value in the base; and while he has devoted 25 years to the use and study of Portland cement and its products, has learned many of its peculiarities, and believes it to be the greatest building material used, he does not believe that ordinary concrete, in street pavements where it receives a most severe test, is as satisfactory as vibrated concrete. He thinks it is paramount that every square inch of such a pavement should have nearly the same strength, texture and abrasive resistance.

THE MANUFACTURE OF CONCRETE PRODUCTS IN GERMANY.

By Julius Carstanjen, Jr.*

The artificial stone industry in Germany, which depended on the development of the cement industry, can be traced back for about 50 years. The cement at that time was quick setting, which made it possible to use the forms repeatedly, at short intervals. The method of manufacture consisted in casting the concrete, of very wet consistency, in wood and plaster forms. All kinds of architectural and ornamental stone have been made in that manner. The surface, however, appeared flat and soon it became the practice to wash it with water or treat it with acid, whereby the different grains of sand became visible and the appearance of the surface less flat. By using quick-setting cement, however, there was danger of hair cracks which gave the stone a poor and unfinished surface, and this especially occasioned many objections from the layman.

The further development of the Portland cement industry in the manufacture of slow-hardening cement brought about a change in the manufacture of cement products. In place of wet concrete, one of rather dry consistency was adopted that is in use today, being well tamped into the iron, wood, plaster or glue forms. By mixing with the concrete good weatherproof mineral colors, all imaginable shades can be obtained to meet the requirements of the architect. This led to the use of a facing of a rather rich concrete containing only fine aggregate, backed up by a coarse concrete. Colored stone manufactured in this manner is either used as it comes from the forms, or it is washed or treated with acid.

At the end of the last century a further improvement in the production of artifical stone took place, viz., tooling the surface. There can be no doubt that this gives the stone more life. This led to another very important step, that is the use of colored aggregate; this is accomplished by mixing with the facing an

*Duisberger Cementwaren Fabrik, Carstanjen and Company, Duisburg, Germany.

aggregate having the color of the natural stone it is desired to imitate, and produces an artificial stone which cannot be distinguished from natural stone.

Very much depends, of course, on the use of the proper aggregates, correctly proportioned both as to size and mixture. It is best to use a machine mixer. The tamping of the concrete in the forms is done with proper tools, either by hand or compressed air, and results in a perfectly dense product. The facing is about 1 in. in thickness and the backing is of coarse concrete.

After the stone is removed from the form, which is generally by turning it over in sand immediately after the tamping, it is kept wet and is ready for the stone cutter in from 10 to 14 days. The stone, of course, is reinforced as required and is also ground and polished, as is customary for risers, wainscoting, etc.

The same surface effects can be obtained in the reinforced concrete work as with the unreinforced work. In erecting, let us say, an artistic portal, a concrete facing is first tamped in the casing to the customary thickness of 1 in., and by properly tamping the coarse concrete is combined with it; the proper aggregates for the facing gives the surface the desired tone, requiring only the work of the stone cutter on removal of the forms.

Artificial Stone.

Recently some very interesting work was done by E. Schwenk of Ulm, one of the most prominent firms in Germany manufacturing artificial stone. One of their undertakings was the restoration of the Königsbau (Kings' Building) at Stuttgart, ornamented by a long row of Doric and Corinthian columns of fine sandstone (Fig. 1) and erected about 50 years ago by King William I of Wurttemberg. The lower part of some of the columns has been affected by the weather and partially destroyed by spalling off to a height about the level of the eye of the passer-by. The sandstone itself was quarried in the neighborhood of Stuttgart. The officials in charge of the building endeavored to repair the worst places with cement. This, however, did not stop the deterioration of the remaining sandstone. In order to preserve the beauty of the building and to insure its permanence the authorities found it necessary to take radical steps which resulted in replacing the damaged sandstone bases with artificial stone.

Figure 2 shows the method of removing part of the sandstone column bases which are then replaced by concrete blocks. The completely repaired columns may be seen at the left of the picture; evidence of the justification of replacing the damaged sandstone with artificial concrete stone is afforded by the fact that work two years' old is in perfect condition. The manufacturer of the artificial stone was able to so closely match the color and texture of the sandstone that one can hardly note the difference. The artificial stone sections will also be noticed in the new concrete

FIG. 1.—KÖNIGSBAU, STUTTGART, GERMANY; ERECTED BY KING WILLIAM I OF WURTEMBURG.

block column bases; the fresh joints are distinctly visible and gives an idea of the extent of the repairs.

Another work of restoration was that of the statuary on the Royal Palace at Stuttgart, finished about 1750 by the splendor-loving Duke Karl of Wurttemberg. The rich roof balustrade of artistic statuary, almost double life size, is made of sandstone (Fig. 3). A number of years ago these sandstone figures were replaced one by one by replicas in the best kinds of natural stone, such as the famous Kehlheimer and other excellent stone, but at the present time these are also in need of renovation; copper wiring was used to prevent single parts, such as arms, hands, etc., from falling and injuring people below. The danger was so great

that this method had to be adopted, although it did not add to the beauty of the statuary, and in spite of all precautions small pieces constantly fell.

The officials responsible for the statuary came to the conclusion that there would always be this danger as long as sandstone was used, and that not only would the surface crack and spall off but even whole sections drop down. The deterioration of some of the figures only thirty years old was rapid and all possible methods were tried and experiments made to determine the best way to protect the new statuary but nothing could be done

FIG. 2.—SHOWING METHOD OF RESTORATION OF THE COLUMN BASES OF THE KÖNIGSBAU.

to counteract the effect of the weather. Among some of the rather drastic means employed was the replacement of the upper parts of some figures with new parts of sandstone, but the lower portion of the figure continued to decay and it was evident that the upper part would follow the same course in due time.

The officials decided to effect a final remedy and to replace all of the statuary with artificial stone of the same profile. This decision was arrived at after inspecting several old artificial stone figures, which, without any special methods of preservation, had shown practically no deterioration after some 15 to 20 years of

exposure to the elements. It was decided to have the new statuary of the same color and texture as the sandstone and the work of replacement of the statuary was conducted along the following lines:

The old sandstone figures were removed one by one from the building with the greatest care and shipped by rail to the factory. They were so brittle and so deteriorated that they could only be shipped in sections which were reassembled in the factory with great difficulty. This is well illustrated by Fig. 4A, showing a knight about 10 ft. high, originally cut out of a 135 cu. ft. block

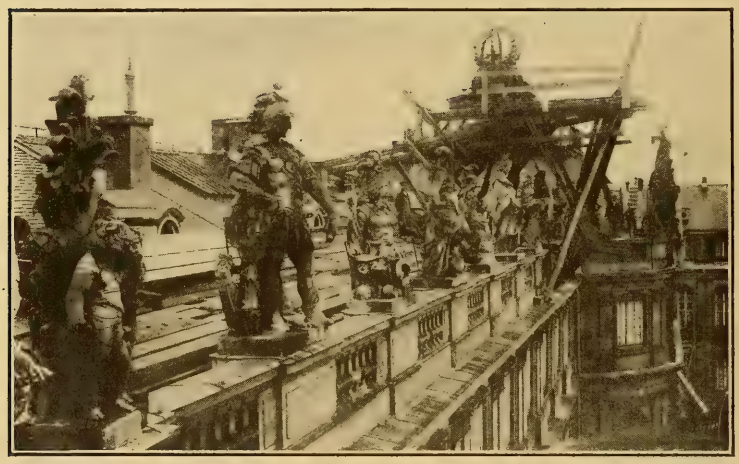


FIG. 3.—SHOWING STATUARY ON BALUSTRADE, ROYAL PALACE, STUTTGART, GERMANY.

of sandstone. The missing and damaged parts have been modeled partly in cement and partly in plaster. The extent of some of these replacements is indicated where the repairs appear in white.

The remodeled statue was used to prepare the plaster and glue forms which were to be employed in casting the new artificial stone statues. After these forms were prepared and placed in position for casting, all parts were adequately reinforced with metal, carefully placed to insure maximum rigidity. After the concrete had been cast and sufficient time allowed for hardening the forms were removed in pieces (Fig. 4B), and so well was the

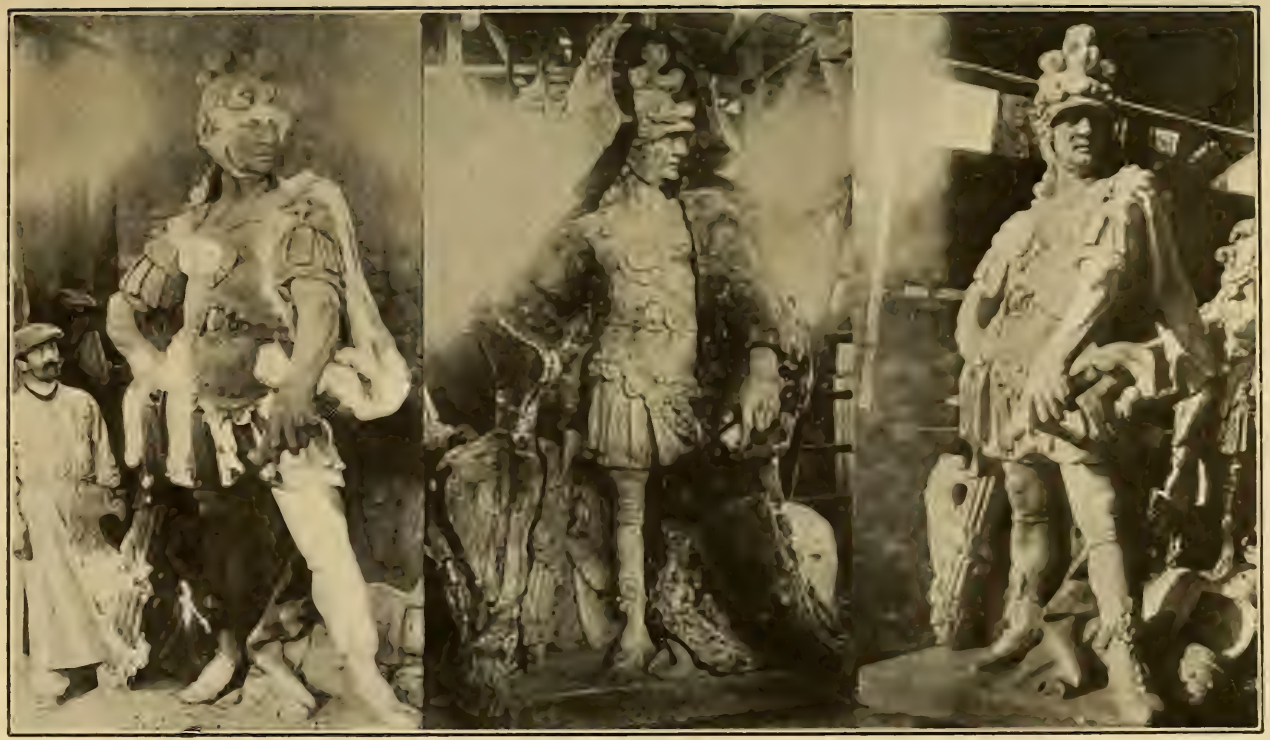


Fig. 4.—Showing method of restoring sandstone statuary deteriorated by weather, from balustrade, royal palace, Stuttgart, Germany.

work done that there was necessary only a few finishing touches of the sculptor who chiseled off a thin layer; figure 4C shows the finished artificial stone statue in the foreground. The difference between the old and new statues is certainly very pronounced in favor of that of artificial stone. It might be of interest to state that it took on the average from 10 to 14 days to complete one piece of statuary.

CEMENT PIPE.

Cement pipe is quite generally used in Germany. The general length of a section is 1 meter (39 in.). Circular pipe is manufactured in diameters from 100 to 2,000 millimeters (4 in. to 80 in. diameter). Egg-shaped pipe is made in a cross-section of 200 by 300 millimeters to 1,000 by 1,500 millimeters (8 by 12 in. up to 40 by 60 in.). Circular pipe is made with a base and also quite generally without a base.

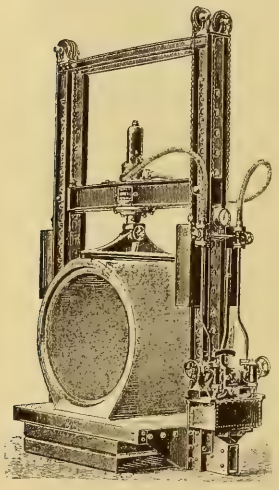


FIG. 5.

The method of manufacture differs according to the size of the pipe and the experience of the manufacturer. Concrete of a rather dry consistency is worked with hand tampers (mostly of iron), but air tampers are also used; others use automatic tamping machines or presses. The stripping of forms in the case of small pipe occurs right after the tamping, but with large diameter pipe it is best to leave the concrete in the forms for some time. The tamping of a pipe is generally done in a large room where the pipe remains for about a week for curing, the time depending on the atmospheric conditions, before being exposed to the wind and weather. In the casting room, it is, of course, very necessary to repeatedly wet the concrete. At least two months and generally three months elapse before the pipe is used.

The method of joining pipe generally used, is to fill the joint of the socket type with a 1:1 or 1:2 mortar; the extending lip of each succeeding pipe is placed at the bottom and surface is covered with mortar before the next section is laid.

Figure 5 is a general view of a hydraulic testing machine with a maximum power of 10,000 kilograms (22,000 lbs.). The load is applied at the top of the pipe on an area of 5 millimeters wide by 1,000 millimeters long ($\frac{2}{10}$ in. by 39 in.). The pipe itself rests on a layer of sand kept in place by a wooden frame. A few results of compression tests of plain pipe of the usual proportions of 1:2:4, good clean concrete 2 to 3 months old, made on this machine will possibly be of interest. The values given are in each case the total load at failure and the results refer to tests with a free application of load on top of the pipe.

CIRCULAR PIPE WITH BASE.

20 in. diameter	9,600 lb.	on 39 in.	(1,230 lb. per sq. in.)
28 "	9,800 "	" " "	(1,257 " " " ")
32 "	9,000 "	" " "	(1,154 " " " ")
40 "	8,000 "	" " "	(1,126 " " " ")

EGG-SHAPED PIPE.

12 x 18 in.	8,600 lb.	on 39 in.	(1,103 lb. per sq. in.)
16 x 24 "	9,600 "	" " "	(1,230 " " " ")
20 x 29 "	12,600 "	" " "	(1,616 " " " ")
32 x 48 "	14,800 "	" " "	(1,898 " " " ")

The usual average water pressure guaranteed is from 40 to 60 lbs. per sq. in. One is, of course, able at the time of the manufacture to reinforce the pipe as desired.

DISCUSSION.

Mr. Watson. MR. CHARLES D. WATSON.—Mr. Carstanjen, is it a common practice in Germany to tamp concrete; and is what we call a relatively dry mix used? I would also like to know the largest and smallest size of the aggregate used in the artificial stone? What treatment is given to the exposed surfaces; are they tooled or do you expose the aggregate by scrubbing?

Mr. Carstanjen. MR. JULIUS CARSTANJEN, JR.—It is the practice to tamp the concrete using a rather dry mixture. The aggregate varies in size from 1 in. down to $\frac{1}{4}$ in. for the body and very much finer than $\frac{1}{4}$ in., in the proportion of 1:2 or 1:3, for the surface of the stone. It is the general practice to polish the surfaces, which is done as soon as the stone is sufficiently hardened to stand the rubbing. Scrubbing is also resorted to to expose the aggregate and a great deal of the stone is finished by tooling, either by hand or with a pneumatic tool. No waterproofing is used in the stone and it is the practice to use only cement that is at least one month old, otherwise there is likely to be trouble from hair cracks. This trouble is also experienced if a concrete is used too wet or is too rich in cement. It is for this reason that a dry mixture is used and very little trouble of this character is experienced.

The President. PRESIDENT HUMPHREY.—It might be well to point out that the dry mix to which Mr. Carstanjen refers is not the dry mix as we understand it in this country. The German method is to resort to a great deal of mixing, and a dry mixture becomes comparatively plastic as a result of this prolonged mixing; in other words, they eliminate the air and excess water voids by thorough mixing and the resulting paste, while stiff, as contrasted with our wet mixture, is nevertheless plastic, which accounts for the water-tightness and density of the product.

Mr. Watson. MR. WATSON.—In the manufacture of deep sections of pipe do you find difficulty in tamping this dry concrete around the reinforcement, so as to secure a smooth surface?

Mr. Carstanjen. MR. CARSTANJEN.—No difficulty is experienced in compacting the concrete in the manufacture of pipe, whether it is tamped

by machine or hand. Most of the tamping is done with a pneumatic tamper; the tamper being raised and allowed to fall by its own weight. We have experimented with vibrating concrete but this is not in use in Germany. Mr. Carstanjen.

MR. C. W. BOYNTON.—I would like to have a description of the type of mixer used and how it is operated in order to produce this dry mixture. Mr. Boynton.

PRESIDENT HUMPHREY.—The mixers generally used in Germany are of the pug mill type. The mixing is secured by thoroughly rubbing materials together. The President.

MR. WATSON.—I would like to ask Mr. Carstanjen if steam is used in curing the cement products. Mr. Watson.

MR. CARSTANJEN.—Yes, it is used to some extent, but it is not a common practice; my firm does not use it and I have had no experience with steam curing. Mr. Carstanjen.

MR. WATSON.—I would like to inquire whether or not the artificial stone that was used to repair the columns that Mr. Carstanjen has described was molded as a unit at the site or made at the factory and shipped to the place where the repairs were being made. Mr. Watson.

MR. CARSTANJEN.—All the material for repairing the building referred to was made at the factory, and after it had properly hardened was shipped to the place where it was to be used. It sometimes happens that buildings are faced with artificial stone in place, although this method is not used to any extent. In such cases the work is generally reinforced. In most of the larger pieces of artificial stone more or less reinforcement is used. Mr. Carstanjen.

MR. EDWARD GODFREY.—I would like to ask whether the concrete is cast in sand or wooden molds, whether the material is kept wet during the hardening and how long this wetting continues? Mr. Godfrey.

MR. CARSTANJEN.—We use plaster and wooden molds, and after they have hardened sufficiently these molds are removed and the concrete covered with sand and kept wet for eight days. Mr. Carstanjen.

MR. WATSON.—I would like to ask whether it is the common practice to core out large pieces of artificial stone, so as to reduce the weight, and if so, how thick are the walls of such sections. In this country we find that the wide ranges of temperature are such that a 1-in. facing of concrete, after ten years, will crack, as a result of expansion. Mr. Watson.

Mr. Carstanjen. MR. CARSTANJEN.—It is the practice to hollow out the larger sections of concrete, so as to reduce the weight. Where a veneer is used it is generally about one inch thick; no trouble is experienced from cracking even after a long period of time.

Mr. Larned. MR. EDWARD S. LARNED.—I would like to ask Mr. Carstanjen whether I understood him to say that they do not waterproof artificial stone in any way.

Mr. Carstanjen. MR. CARSTANJEN.—That is correct. We do not use any method for waterproofing our artificial stone.

Mr. Watson. MR. WATSON.—Are there any specifications covering the strength and density of artificial stone, if so, will you kindly state them?

Mr. Carstanjen. MR. CARSTANJEN.—There are such specifications and they require that artificial stone shall have a compressive strength of 1,700 lbs. per sq. in. after 28 days.

THE COLORING AND TEXTURE OF CONCRETE PRODUCTS.

By Adolph Schilling.*

When, ten years ago, the speaker started to make concrete attractive and pleasing, he accepted as definitely settled its claim for strength and endurance, so thoroughly demonstrated by the engineers of the time and the examples of ancient concrete work still in existence.

His researches and experiments have been devoted entirely to producing in concrete those artistic effects which would make it interesting to the architect and decorator as a medium for the embellishment of their work. If for this reason these remarks are of less interest to engineers and contractors in concrete construction than more technical matters are, their indulgence is asked, to consider for a few minutes concrete as something which is as pleasing from the esthetic as from the utility point of view.

Twenty years' practical experience with the natural stones used in monumental and building work enabled the speaker to appreciate the conditions that must be met to give concrete a place beside the limestones, marbles, granites and metals, the almost exclusive medium by which architects have expressed their thoughts.

After once thoroughly understanding the qualities of cement as a binder or matrix, one can learn to use the many mineral and metal products as readily as the better known sand, gravel and crushed stone are used.

To produce color effects, the gray and white Portland cements may be used, either by themselves or mixed in certain proportions; adding suitable pigments; in many cases, the natural colored aggregates, sand, silica, pebble grits, marble and granite will give excellent and more uniform results; it requires great skill and care to properly mix cement and color pigments, so as to retain the strength of the cement and obtain uniform color

* 908–910 Chestnut Street, Philadelphia, Pa.

effects. The importance of mixing the pigment thoroughly with the cement before adding the aggregates should be recognized by anybody attempting to make concrete in colors.

As a very simple method of testing the proper amalgamation of the pigment with the cement, take a handful of the mixture and press it under a sheet of stiff paper, this will produce an even surface of the material, and as long as this surface does not show absolute uniformity in color, the mixing is incomplete. The small specks of unassimilated pigment that show in this test surface will also appear in the concrete.

Some excellent work of modeling in cement has been done recently by artists who attained their efficiency abroad; the most important work of this kind, to my knowledge, is a group of the crucifixion of heroic size at Lynn, Mass. The artist builds up his design in cement from a sketch model or cartoon, and is able, by using a mixture of quick-setting and slow-setting cement, to regulate the setting of his medium; in this way he can give the work all the freedom and spirit of work modeled in clay, producing any texture desired; color effects can be obtained by using colored cements of various shades; for instance a garland of flowers can be modeled in the natural colors; there is no difficulty in producing the well-known Della Robbia effects in cement, as is done in terra cotta. This method is particularly interesting and valuable where only one piece of a large ornamental design is to be used, as it eliminates the expense of molds and erection in place; and prevents unauthorized replicas of original designs.

A very important factor that can be readily utilized to give color to concrete or cement has been given little or no attention; while very simple in its primary action, its successful application requires thorough understanding of the principle and medium employed. The absorptive qualities of concrete during its stage of curing and seasoning offer opportunities for coloring its products through this capillary action. The color is deposited in the pores of the surface, amalgamating permanently with the concrete. The possibilities of this treatment are unlimited, a knowledge of the coloring values and judgment in its use so as not to injuriously impair the strength of the concrete being essential to success. A coloring solution can be made to penetrate the surface of concrete 6 in. or more, if the object is placed in the

solution in a very green state; it is rarely necessary to penetrate more than from $\frac{1}{32}$ to $\frac{1}{8}$ in.; this thoroughly fills all pores, gives the desired color effect and is less expensive. Every atom of coloring absorbed by the concrete reduces the strength of the solution; and as some of the colors used are quite expensive, the use of good judgment in allowing only the necessary absorption of coloring matter is advisable from an economic point of view. Solutions of aniline and the sulphates of copper and iron are the most suitable for coloring concrete by the capillary method. The concrete can be treated after it is several days old; concrete products requiring considerable strength should not be subjected to the coloring bath until they are sufficiently hardened, since the filling of pores stops the action of curing by the usual methods.

Coloring by absorption is effective on surfaces of concrete immediately after it is removed from the mold, or after treatment with acid, or tooling. Surfaces colored through the absorption of mineral or metallic colors become waterproof, and the action of weather on metallic color is the same as on metals, increasing the beauty of the coloring by the same oxidation that occurs on bronze and copper. The surfaces of concrete treated by this method become so hard and dense that they will take a uniform dull or high gloss polish. The speaker has treated such surfaces with polishing and buffing machines, the same as marble, granite and metal are treated.

Products made by the speaker's methods, such as flower pots, vases and boxes, will hold water the second day after casting, and become so hard that when struck with a hammer they ring like a bell; he does not consider the use of waterproofing compounds essential to obtain this result, but considers the proper amount of water and thorough grading of aggregates as all-important.

The speaker has made extensive tests during the past three years with waterproof paints, and has obtained excellent results. Concrete can be made very attractive with one or two coats, and if applied by the stipple method, it will not impair the grain or texture and avoids the undesirable effect of painted stone. This method is especially recommended for dry or semi-dry process concrete work, as its porous surface readily absorbs the waterproof liquid and allows the pigment particles to fill the pores. The

color effect obtained in this manner is the most economical, but it is a flat color and does not give the richness and depth of shading that result from the immersion treatment in an aniline or metallic color bath. It can be used to advantage, however, where immersion is not practicable.

The speaker has obtained excellent two- and three-color effects by painting certain parts of an object before subjecting same to the coloring bath, the parts so colored not being affected by the color in the bath. The artistic possibilities of such treatment are only limited by the color sense and taste of the craftsman.

By using certain non-absorptive aggregates, their natural color can be retained, while the absorptive part will assume the desired color. In this treatment precaution must be taken not to use certain acids in washing before immersion in color bath, as the chemical action of the acids is liable to counteract the color value of the bath.

Practice and testing will give the experience necessary for gaining the full benefit of this process; the speaker has been ten years in obtaining his present results.

In the matter of surface finishes, considerable headway has been made, and most of the methods are well known to the progressive users of concrete.

Concrete of proper age can be treated like any natural stone, using the same tools and machinery to dress its surfaces; it is the speaker's strong conviction that the success of concrete stone for building purposes rests with a close affiliation of the stone caster and the stone cutter; only in this way will we be able to give concrete proper texture and the necessary qualities of dimension stone so essential to the architect and builder.

For six years the speaker's concrete stone and natural stone plants were under one roof, and these conditions were found to be ideal in procuring first-class composite stone of proper size, quality and finish.

The addition of a skilled stone cutter and setter to every concrete products plant would be a source of profit and result in better work; he knows what it means to have dimension stone true to size and shape; the average worker in cement or concrete does not appreciate this essential point, which is all-important to the architect and general contractor. The services of a stone

setter will enable the concrete products plant to set its own work, which is very necessary for good final results.

The brick or natural stone mason, for reasons of his own, does not give composition stone the required care when setting it. But once give this fraternity to understand that concrete stone means employment to stone cutters and setters, and its antagonism will vanish. Whenever the opportunity affords, the speaker strongly recommends closer affiliation of the composite and natural stone trades. The enormous increase in high-class building operations and the constant effort for betterment in the moderate-priced homes give abundant fields in both trades for work of a better class and at a more satisfactory profit.

The speaker has found that every machine, tool and implement now used in finishing natural stone, can be employed to good advantage in dressing concrete products. It has taken many years to perfect the business of dressing natural stone; the progressive concrete products man should study these methods and benefit by the experience of the man whose product he must equal or better to win the approval of the architect.

The speaker further recommends to the manufacturer or worker of concrete products, the adoption of such methods as are employed in other lines of manufacture; to treat the surfaces of their product, for instance, instead of applying paint with a brush, immerse it in the paint; this immersion is specially adapted for acid washing. It should not be a vital matter to construct tanks of sufficient size in a concrete plant, and the soaking of the surface of concrete in acid will not only prove a great saving in acid, but will produce a class of work that cannot be obtained with the scrubbing brush; a very weak solution is required, the treatment preserves the edges and details of the design, and makes the surface uniform. Any of the hard spots not sufficiently affected by the acid bath can be treated separately after the article has been flushed with clear water. Care must be taken that the aggregates of the surface are of nearly uniform hardness, or the acid will eat the softer portions before the harder particles have been cleaned of the coating. For instance, the speaker had some very fine work spoiled by this treatment; to obtain a certain effect, a mixture of black marble (a limestone) with crushed granite was used; the acid bath left void places where the black

marble had been, while the granite showed a very fine texture and natural color.

For the encouragement of collaborators in this field, the speaker presents the following abstracts from letters received from two prominent New York architects by him in 1909, that express the opinion of many as to concrete stone:

From ALFRED HOPKINS:

"The gates (at Great Neck, L. I.) have been admired very much and I don't think I have ever thanked you for your personal interest in the matter and your willingness to carry out the details of the work, which were considerably more elaborate than you had reason to expect from the working drawings. As I told you then, I am very much interested in this concrete work, and am certain of the future of it.

"There is no doubt in my mind that the architects are becoming more and more interested in ornamental concrete work, and many are using it in preference to stone work, Mr. Goodhue, of Cram, Goodhue and Ferguson, who build a great many churches (Mr. Cram having recently been appointed architect of the Cathedral), tells me that they use concrete entirely for the Gothic traceries in the windows, and concrete costs a very little more than wood, and about one-third of stone.

"I have made a specialty of concrete work for the past six or seven years, and I know that the interest, not only of the profession, but of the public, has greatly increased in such matters, and there is no doubt in my mind but that field is broadening every day."

From JULIUS HARDER:

"I am among those who believe that composition concrete is destined to occupy an ever-increasing field in the progress of the building industry. Although at the present moment the material is still in an experimental stage, its future is attractive as well as extensive. It is understandable that some attempts and trials have resulted in failure, but successful instances are also plentifully recorded.

"Artificial composition material should not be employed in a manner imitative of its natural counterpart. Such misuse restricts its scope and creates prejudice against it. I think a great deal

of the feeling of opposition to concrete stone is the result not so much of defect of the material itself as of wrong employment in design and in the manner of use. With the correction of this feature at the hands of designers, the plain merits of the material will become apparent and be accorded deserved recognition. The most important of these merits consist of the greater resistance of the concrete stone to fire and moisture, and the consequent greater durability and cheapness.

"Artificial and concrete compositions must possess all the merits of natural materials, and possibly some in addition, without the corresponding defects. Your material answers to this requirement to an extent beyond expectations, and I am confident that your work in this field has advanced the status of composition stone in the building art materially."

As a closing word, while not assuming to instruct the architects present or elsewhere, the speaker wishes to advocate the most extensive use of properly made concrete stone.

The use of crushed marble and granite, with Portland cement as a binder, producing a reconstructed stone of natural components, is not an "imitation" of natural stone or a "misuse of concrete;" but it gives us a staple building material, combining the beauty of color and texture of the natural stones with the strength and economy of concrete.

The speaker's object in presenting this paper to the concrete fraternity is not altogether unselfish; he expects that with many heads working along similar lines, new features and betterments will result; an exchange of ideas is solicited by him.

REINFORCED CONCRETE SILOS AND SMALL GRAIN BINS.

By E. S. Fowler.*

The year 1912 has been one of the greatest silo building seasons in the history of this country. Iowa, so far as we can learn, has taken the lead in silo building, having completed 10,000 silos this year; one township alone has 58 now in use. Last year St. Croix County, Wisconsin, and Kane County, Illinois, had 125 and 150 silos in use, respectively.

A silo may be defined as an air-tight receptacle for the storage of forage to preserve it in a green and succulent stage for the feeding of stock during the winter months and summer droughts. The term silage denotes the green forage preserved in the silo. In the corn belt, cornfodder is almost exclusively used, while other sections use alfalfa, clover, pea vines, cow peas, etc. Many people think of the silo as something new, and thousands of farmers have yet to see their first silo. The use of what is now called silage, however, dates back many hundreds of years. The process of preserving leaves and grains was practiced by the early Egyptians, the American Indians, and by the Aztecs of Mexico. Varro, an early Roman traveler, tells of the use of silage in Rome, Carthage, Thrace and Spain. However ancient as this process is, the method of keeping silage was not unknown to the farmers of several nations at least one hundred and twenty-five years back.

The early silos were very crude. Leaves and grains were packed in jars and other casks not unlike the modern suaerkraut. Then some more enterprising agriculturist conceived the idea of digging a pit in the ground and preserving succulent food in that manner. These pits were later lined with cloth, wood or brick. The first pit known of had a maximum depth of 12 ft. The first "above ground" silo is credited to M. Gouffet of Solyne, France. He was a very enterprising man, studied silage carefully, and in 1877 published a book on the subject giving his experience. The

* Assistant Engineer, Information Bureau, Universal Portland Cement Company, Chicago, Ill.

first "above ground" silo in America was constructed about 1875. Soon after, a concrete silo was built by a Mr. Bailey of Massachusetts. In 1882, only 92 farmers could be found using silage with success. Later, Dr. King, of the Agricultural Station, Wisconsin, took up the study of silage and made a thorough investigation, and with large publicity perpetuated the use of silage in this country.

Most of the first silos in America were built of stone and were very rough. This led to a large number of experiments with brick and wood. Cement blocks, clay blocks and monolithic concrete next appeared on the market. However, the pre-requisition necessary for a perfect silo has placed concrete foremost as a silo material. Investigations and favorable recommendations by the leading agricultural experiment stations in this country have placed concrete at the front as a building material for silos.

The demand for silos is not limited to farmers, but the large canning factories are now finding it a valuable asset in utilizing the waste products of canning, such as pea vines, corn husks, etc.

The demand for silos among the farmers is increasing at an enormous rate. The dairy farmer finds that silage is the most economical succulent food that can be obtained for dairy cows, and when fed with legume hay, such as alfalfa or clover, it provides the nearest approach to green pasture yet discovered.

Prof. Heacker, of the University of Nebraska, makes this interesting statement: "An acre of corn put into the silo I value at $55 while the same standing in the field and husked in the usual manner I value at $27. This is accounting for all costs of harvesting. Then an acre in the silo is worth two in the field, or, putting it another way, the silo doubles the value of the corn crop." Mr. A. D. Wilson, Superintendent Farmers' Institutes, Minnesota, estimates that without a silo 80 per cent of the cornstalks are wasted. Mr. Charles Nelson, dairy expert, of Atwater, Minnesota, also states that it costs 14 cts. more per pound to produce butter without a silo than with one.

Mr. H. M. Cottrell, Agriculture Commissioner for the Rock Island System, has compiled some interesting figures along this line upon the appalling waste in the corn belt. He states:

> One of the greatest, if not *the* greatest, waste in any single industry in this country is made with the corn crop. The grain of the annual corn crop of the United States is worth on the farm one and a half billion dollars. In a

ripe corn plant, 60 per cent of the feed value is contained in the kernels and 40 per cent in the stalk and leaves. With the grain worth on the farm one and a half billion dollars, the feed value of the rest of the crop, if fully utilized, is a billion dollars. At least 90 per cent of the feed value of the stalk is lost under the present system of farm management—a waste with this crop alone of nine hundred million dollars yearly. No other business but farming could stand such an enormous loss.

Reasoning from the authoritative statements above, we conclude that no dairy farmer can profitably produce dairy products without a silo. Furthermore, where a farm is such that all parts are tillable, a farmer cannot afford to assign part of his farm for grazing purposes. It takes two acres of good pasture to graze one cow during the summer months. If one acre of this is put into corn it will increase its value to two acres in the silo, or eight tons of silage. Therefore, enough corn can be produced from the grazing land used for one cow during one season to make enough silage for four cows for the same length of time.

Realizing the great economic value of the silo for dairy farmers, several of the leading agricultural colleges have recently carried on experiments which substantiate the absolute necessity of the silo in beef production.

Our western ranches are rapidly dwindling away and the stockmen in the far West who have found cattle raising in the past profitable by grazing now are finding their profits less on account of the homesteaders gradually taking up claims and diminishing the grazing area. A large ranchman in Western South Dakota recently bewailed the above conditions, and stated he was compelled to winter several hundred two-year-old cattle on Russian thistles last winter because of a shortage in his corn crop due to the drought in the summer. If he had been equipped with silos, with a half crop, he could have easily carried his cattle over in excellent condition. This gentleman stated it was his intention to build ten silos for winter feeding. Many a farmer under similar conditions has been compelled to make disastrous sacrifices in disposing of stock (in many cases well-bred stock), because of droughts and feed shortage.

The value of the silo established, the prospective builder finds himself confronted, not with the question—Can I afford a silo?—but can I afford to farm without a silo? His next problem is what

kind of a silo to build, and after careful investigation, his decision will be, a silo of concrete. To prove this, let us investigate the requirements of an ideal silo? A silo must be durable, moisture-proof, and have air-tight, strong, rigid, smooth walls.

It must be moisture-proof and air-tight to prevent moisture from passing out, and air from coming in, because oxygen in the air induces fermentation and causes spoiled silage. Silage can be compared to the keeping of fruit in air-tight receptacles. Germs which produce fermentation can only exist in the presence of oxygen. Therefore, with small particles of green forage, well

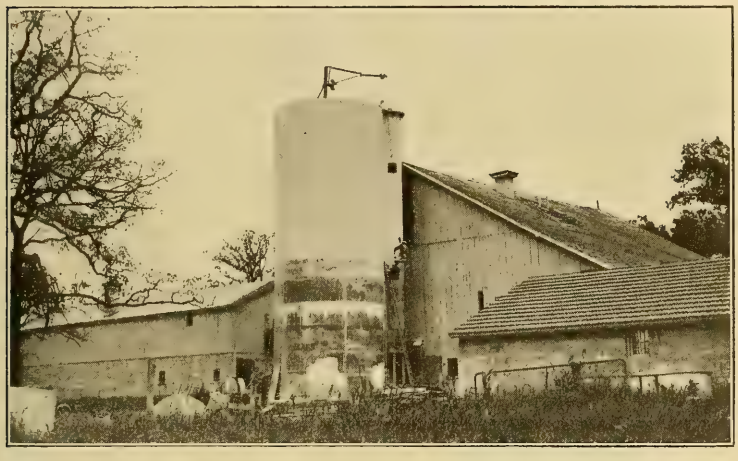


FIG. 1.—CONCRETE SILO, 12 FT. DIAMETER 40 FT. HIGH, RECEIVING FINISH COATING, HARTWOOD FARMS, BARRINGTON, ILL.

packed in an air-tight moisture-proof silo, the extent of fermentation will only be according to the amount of air admitted when filling.

It must have strong, rigid walls to insure them from being crushed at the bottom of the silo, due to the weight of the silage, also to resist the bursting pressure of the silage in the silo, and to withstand the pressure of heavy winds when empty. The walls must be smooth to insure the perfect and easy settling of the silage so that air pockets will not be formed around the sides.

The reinforced concrete silo (Figs. 1, 2 and 3) qualifies in all of these requirements. In addition to the requirements of an ideal

silo, just outlined, the concrete silo is low in its first cost of construction with no maintenance or upkeep and its pleasing appearance places it first in all types of construction.

In building a silo, the location should be such that it can be connected with the barn so that there will be easy access to it in

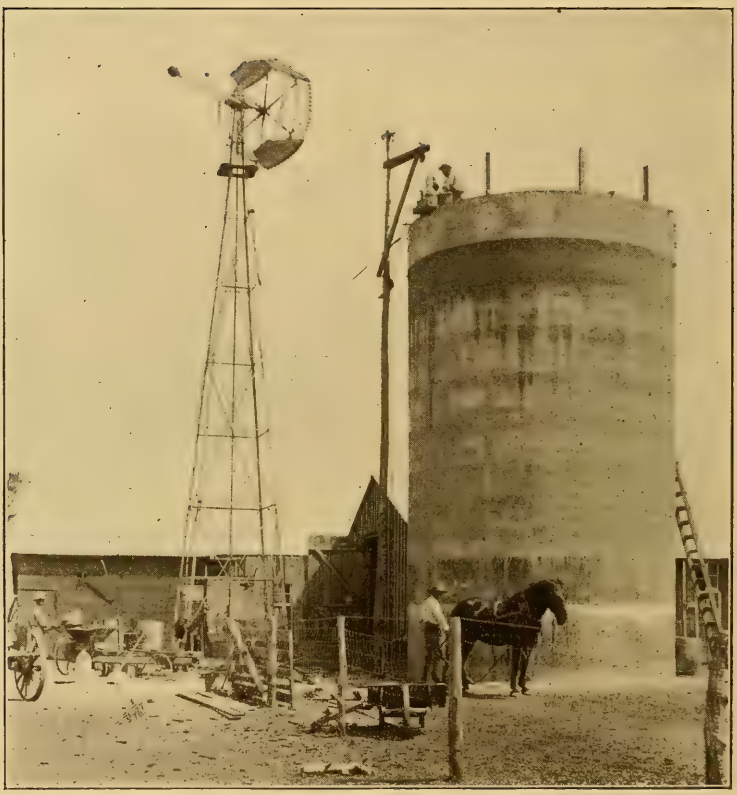


FIG. 2.—REINFORCED CONCRETE SILO CONSTRUCTED BY GREEN BROTHERS, FARMERS, MORGAN, MINN., WITH HOME-MADE FORMS.

conveying the feed to the feeding troughs. If a monolithic silo is to be built, care should be taken to secure good forms which will operate easily and permit the maximum amount of concrete to be placed each day. Excavation for the foundation should be made a foot wider than the diameter of the silo to permit the form being

started at the bottom of the silo and thus facilitate keeping them level throughout the construction. All silos should be provided with concrete chutes to protect the doors through which the contents are emptied, and with roofs to keep out the storm and prevent freezing of the top of the silage. Floors are essential to keep water from entering the silo and the silage from being pressed into the

FIG. 3.—REINFORCED CONCRETE SILO, 16 FT. DIAMETER BY 60 FT. HIGH; 400 TONS CAPACITY WITH 18,500 GAL. REINFORCED CONCRETE WATERING TANK. TOTAL HEIGHT 79 FT.

dirt. When desired, a water tank can be placed on top of the silo, which will furnish water where needed about the farm.

What has been said about silos will apply equally well to small reinforced concrete grain tanks (Fig. 4) for farm use. The early grain bins used by farmers were of wood, made in the shape of small buildings. These were usually located some distance from the other

buildings for fire protection, which made feeding very unhandy. Such construction depreciated quickly and gave little or no resistance to the invasion of vermin, thus causing considerable loss. All wastes considered, the farmer's experience is that by selling direct

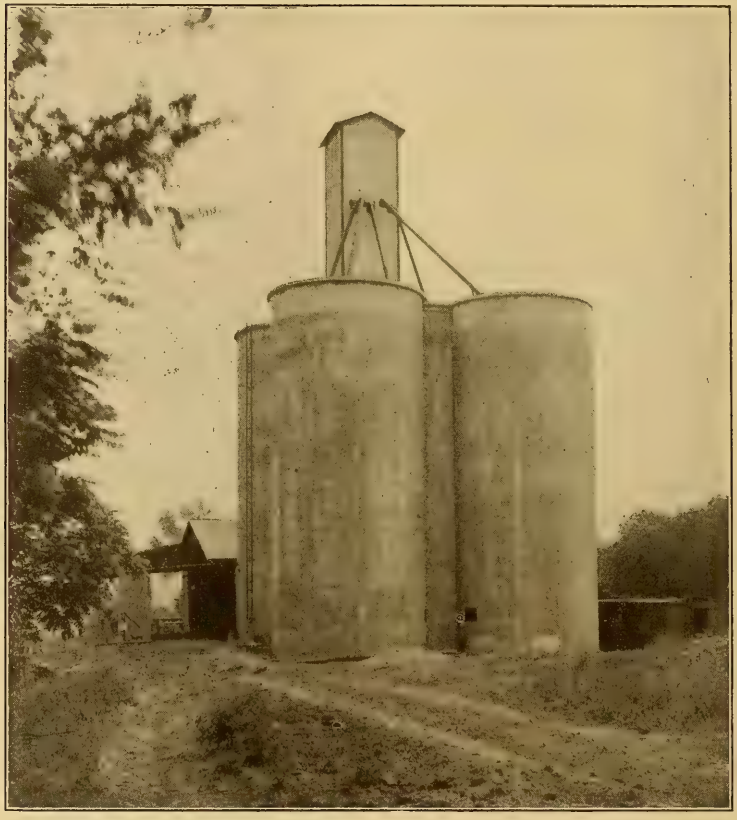


FIG. 4.—MONOLITHIC REINFORCED CONCRETE GRAIN STORAGE HOUSE, ZILIAK AND SCHAEFER MANUFACTURING COMPANY, HAUBSTADT, IND.

from the threshing machine he can take five cents less per bushel than by holding it in wooden bins with their inherent waste and fire risks.

A few farmers in the grain-raising states have taken a step in advance and are making small overhead grain bins in their cribs

and barns, thus reducing the cost of separate buildings. They still have, however, the problems of waste heretofore mentioned and an additional risk of fire. Reinforced concrete grain tanks properly constructed reduce these wastes to a minimum, for they are vermin-proof, moisture-proof, wind-proof, fire-proof and durable. One reinforced concrete bin will hold all the grain raised on the farm.

Statistics compiled by the Chicago Board of Trade show that of the available supply of grains in the United States on March 1, 1912, only 19.6 per cent of wheat, 34.9 per cent of corn, 31.4 per cent of oats, was in the hands of the farmers; also that the highest price paid for these respective grains occurred after this date. This large percentage of grain, disposed of at a disadvantage, is mostly due to the poor facilities for storing the same on the farm.

With proper facilities after the corn has been cured, it could be shelled in the winter months when help is plentiful, and stored for high market. Corn usually reaches its highest price about the middle of May At this time farmers are very busy preparing and planting their next year's crop and it is impossible to secure the usual help of neighbors.

On a large grain farm in Illinois two different years the tenant had between six and seven thousand bushels of corn in his crib that he was unable to move because he could not secure a sheller and help. When it was moved he had to take eight and ten cents less per bushel, a loss very nearly equivalent to the cost of a reinforced concrete bin. If it had been shelled and stored early in the season the owner could have used his farm help in hauling it to market.

The average business-like farmer has never before had better advantages for building permanent farm structures than at the present time. The value of improvements of this character are to be appreciated, not only in this, but in future generations, and are to be measured not only in the saving of dollars, but in the saving of many hours of toil. It need hardly be mentioned that the addition of a permanent farm improvement is an added asset, and has its influence upon the selling price of the property upon which it is located.

The use of concrete in building nearly all classes of permanent farm structures is proving so popular that the farm without such

improvements will soon be the exception rather than the rule. Such changes come about through natural causes and are not the result of the obtrusion of commercial and industrial companies, as some would suppose. The depletion of the forests and the consequent high price of lumber is the most important of these causes. We are ever ready to lay hold upon that which is nearest at hand. The natural inclination, therefore, is to transform the sand and gravel in a nearby deposit into the needed structure, be it a barn,

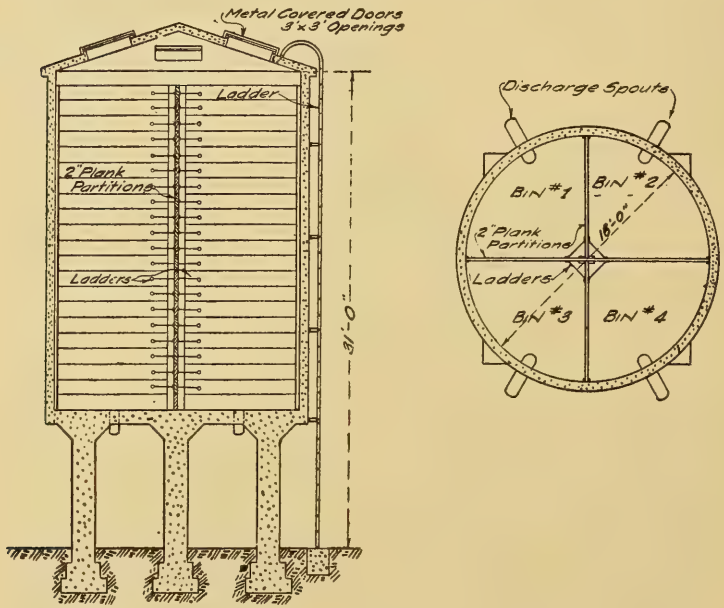


FIG. 5.—REINFORCED CONCRETE GRAIN TANK; CAPACITY 3,500 BUSHELS.

a silo or a fence post. Cement has made this possible, and it remains for us to develop our knowledge, and train our hands to build the needed structure in a substantial manner, and in so doing not to neglect to incorporate such features as will add permanent value to the farm as a whole, and will conserve the farmer's time and lessen his labors.

Since the concrete grain tank is one of the newer farm structures, certain points of interest concerning its construction and cost, and the advantages to be gained by its use will be touched upon.

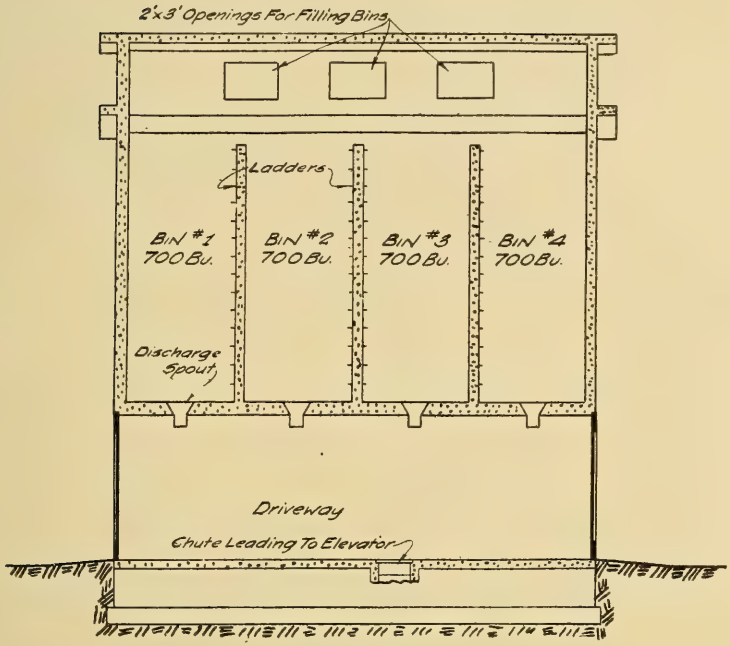


FIG. 6.—REINFORCED CONCRETE GRAIN HOUSE WITH ELEVATOR; CAPACITY 2,800 BUSHELS.

Two designs are submitted as suggestions for small reinforced concrete grain bins for farm use The elevated grain tank shown in Fig. 5 is designed to hold 3,500 bushels of grain. It is elevated

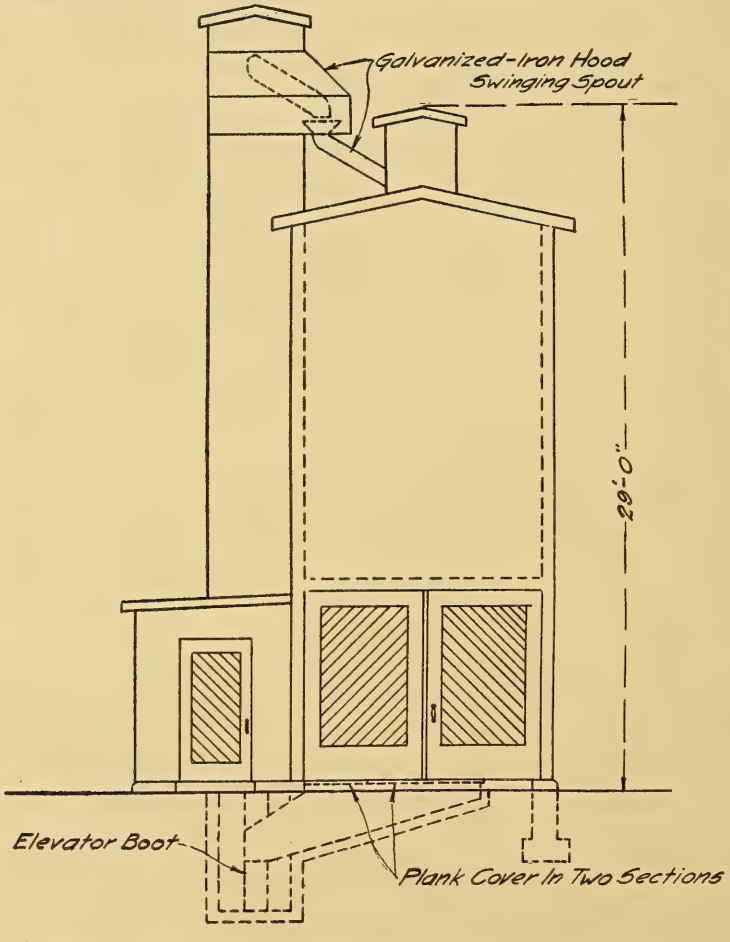


FIG. 7.—END ELEVATION OF REINFORCED CONCRETE GRAIN ELEVATOR SHOWN IN FIG. 6.

about 9 ft. above the ground so that wagons may be loaded from it, by gravity, at any one of the four spouts shown.

The tank has an internal diameter of 16 ft. and a depth of 22 ft. The interior is divided into four bins, as shown, by 2 in. plank

partitions, which are dropped into place, the ends being held by the light steel channels bolted to the walls at the quadrant points. Iron pipe thimbles are imbedded in the tank walls when it is being built to accommodate the bolts which secure these channels. The tank is filled through the trap doors in the roof by a portable elevator which is to be found on nearly every up-to-date farm.

This grain tank corresponds exactly to a silo in construction, except for the fact that it is elevated upon concrete walls, has no side doors or chute, and requires heavier reinforcing, owing to the greater pressure of small grain. Above the floor line of the tank, the customary practice of building silos with steel or wooden forms is followed, the same forms being used for one as for the other.

It is interesting to compare the method of building this tank with a silo of the same dimensions and to note wherein the cost of the grain tank is different from that of the silo. An average of the cost of eight silos, 16 ft. in diameter and 30 ft. in depth, located in three different states, is $3.33 per ton capacity, making the average cost of the silos $400 complete. The bottom of the grain tank, which is to be 9 in. in thickness is built monolithic with the three plain concrete supporting walls which are carried to sufficient depth to give a firm foundation. This sub-structure costs about $200 when cement is worth $1.75 per barrel, and sand and gravel $1.00 each per cubic yard. This price also includes a proportionate part of the contractor's profit.

The circular wall of this tank is of the same thickness as that of the silo, but 760 lb. more reinforcing are necessary, which, at a price of 3 cts. per lb. bent and in place, adds an item of $22.80 to the cost of the grain tank over that of a silo of similar size. It has been figured that the plank partitions and fastenings in the grain tank are even in value with the chute and side doors of the silo, so these items are not considered in this comparison of costs. The roofs of the two structures are alike except three extra doors have been provided so that any bin may easily be filled. These extra openings, with metal-covered doors are estimated to cost $5.00 each, built and in place.

It is evident that the elevated grain tank described is similar in all the essentials to the concrete silo, and can be built as in the present example for about 18 cts. per bushel of storage capacity.

Fig. 6 shows a rectangular house, provided with four bins holding 700 bushels each, or having a total capacity of 2,800 bushels. This grain house is provided with an elevator operated by a small engine for which space has been allowed. A driveway extending the length of the house and enclosed by double doors at each end is provided for receiving the grain. The load of grain is dumped into a receiving hopper below the floor level. This is always kept covered except when the grain is being elevated to the bins above. From the hopper the grain runs into the elevator boot and is distributed to the bins by a swing spout. In case the grain in one of the bins should heat it can be spouted into the receiving hopper and re-elevated to another. One advantage of the arrangement of this building is the facility with which grain, such as wheat and oats, are cleaned and thus made ready for the market or for seed. The fanning mill is located in the driveway and is fed from the bins above by gravity. After being cleaned the grain is again elevated to be loaded out as desired. Thus the whole operation can be accomplished with little labor. The driveway also is used as a place for keeping grass and other seeds which require dry and vermin-proof storage, a place that very few farmers have.

Silos and small grain bins for farm use are in demand. We have mentioned only a few of the advantages of the small concrete grain bins. The past season's experience of car shortage is not to be treated lightly; the frequently over-crowded elevator is to be considered. Prof. R. A. Moore, Wisconsin University, says:

"To realize how fast the silos are going into Wisconsin, I need but state that one of our country agricultural schools in 1910 had the county board appropriate sufficient funds to build six silo forms for the use of farmers in putting up concrete silos. With the six forms last year there were sixty silos built and three hundred farmers were on the waiting list for the forms at the end of the season."

The farmers of this country now realize the economical advantages of the silo and grain tank, and contractors, especially the cement men, should awaken to the vast possibilities and grasp the opportunities presented in this field of construction.

ANNUAL REPORT OF THE EXECUTIVE BOARD.

The Executive Board presents herewith the following report on the condition of the affairs of the Association.

Meetings.—Since the last Convention, the Elective Members of the Executive Board have held one meeting, the Executive Board has held two meetings and its Executive Committee has met at frequent intervals for transacting current business. An abstract of the minutes of these meetings is appended hereto.

Membership.—The membership at the close of the last meeting was 1,092, since that time 96 new members have been added, 51 members have resigned, 228 have been dropped for the non-payment of dues, and the Board regrets to record the death of the following members of the Association:

William J. Genther,	Arthur A. Phlegar, Jr.,
Percy S. Hudson,	Ralph Peverly,
H. A. Maine,	Charles H. Ramsey,

George E. Robeson.

At the present time the membership is 902.

The Board regrets especially the loss of one of its members, Percy S. Hudson, who died November 26, 1912. Mr. Hudson was a quiet worker whose services were thoroughly appreciated and whose loss will be greatly felt in the councils of the Board. The Board wishes to offer the following minute on the death of Mr. Hudson:

Through the death of Percy S. Hudson, a member of the Executive Board, this Association has sustained the loss of one of its most valued members. Quiet, unassuming, with the sincere purpose at all times of giving his best thought and effort to those associated with him, the members of the Executive Board feel most deeply his death and wish to express to his family their most sincere sympathy.

Resolved, That this Minute be spread upon the records of the Association and a copy thereof be sent to his family.

Report of the Treasurer.—

RECEIPTS.

Balance, July 1, 1911		$852.39
Dues	$3,379.70	
Dues, Contributing Members	550.00	
Sale of Publications	2,615.25	
Convention at Kansas City	1,000.00	
Membership Certificates	27.50	
Subscriptions	1,273.00	
Research Fund	902.50	
Interest on Deposits	9.23	
Miscellaneous	5.55	
Total Receipts		9,762.73
		$10,615.12

EXPENDITURES.

Salaries	$2,577.27	
Office Rent	720.00	
Postage, Printing, Stationery, etc	1,485.19	
Meetings of Executive Board	54.25	
Publications	3,164.40	
Membership Certificates	3.30	
Convention Kansas City	1,419.03	
Miscellaneous	59.05	
Total Expenditures	$9,482.49	
Balance, June 30, 1912	1,132.63	
		$10,615.12

NOTE.—Due for bills incurred and unpaid during year ended June 30, 1912, viz:

The John C. Winston Co., for printing Proceedings, etc	$3,069.35
Richard L. Humphrey, for money advanced to pay Winston's bill	1,141.07
Arthur W. Kelly, for reporting Convention Proceedings	324.00
	$4,534.42

PHILADELPHIA, 4th December, 1912.

We hereby certify that we have audited the accounts of the Treasurer of the National Association of Cement Users for the year ended June 30, 1912, and found that the dues collected and receipts from sales of publications, etc., as they appear in the cash book, have been properly accounted for, and that all disbursements were substantiated by vouchers.

The foregoing report of the Treasurer is in agreement with the books of account.

(Signed) LYBRAND ROSS BROS. & MONTGOMERY,
Certified Public Accountants.

Proceedings.—The Proceedings of the Association have increased from year to year until it is apparent that a maximum has been reached and the difficulty now is to keep down the number of papers offered for each annual convention. The standard of the Proceedings is comparable with those of other associations.

The Proceedings of last year's Convention have not been published because the Executive Board felt that the finances of the Association did not justify increasing its debt at the present time. Under "Income" and "Expenses" the Board will discuss the finances, outlining a plan by which sufficient revenue may be obtained to meet the needs of the Association, and if this plan is approved and carried out it will then be possible to publish promptly not only the Proceedings of the last but those of the present Convention. Certainly it is a fact that the Proceedings themselves form a very valuable reference concerning the various uses of cement, specifications and best practice, and it is highly essential that they shall be published, adding from year to year such additional information as the development of the industry may justify. The growing importance of the standards of the Association would seem to justify their publication in a single volume yearly, so that each member may be advised as to the latest specifications of the Association.

Convention.—The great number of valuable papers scheduled at the present Convention, together with the various committee reports, show the active interest which is being manifested at the present time. In arranging a program for the Convention, the Board found it extremely difficult to prevent having too many papers, and was forced, in view of the fact that the program was already crowded, to decline with regret many valuable papers which were offered for presentation. In preparing the program the Board had in mind first, the reports of committees with their recommendations as to best practice and specifications, and then papers contributing original information which the Board wishes to acknowledge with its appreciation, there being several papers presenting data and important information for the first time.

With the view of making it possible for members to stay through the entire time, the Convention period this year has been shortened, and in order to provide an equal number of sessions some are being held in the afternoons. In response to the wish

of many of the members that they might have an opportunity of attending the opening of the Cement Show, no session is held on Thursday evening.

The Board desires to express its appreciation for the hearty co-operation and support it has received from the many speakers and to state that it has been found difficult in a large measure to limit the papers it was possible to care for. To those persons from whom it was found impossible to accept papers, the Board expresses the hope that opportunity may be afforded at the next Convention.

Income and Expenses.—The most serious problem at the present time is the question of income and expenses. During the last few years the progress and work of the Association has been such as to involve expenditures in excess of the income. It has been necessary, therefore, to solicit advertisements in the Proceedings and contributions to furnish additional revenue to meet the total expenses. These efforts have not been wholly successful and each year has seen a gradual increase in the deficit. This has entailed a great deal of work on the officers of the Association which it is felt should not be reasonably expected of them. A partial effort has been made to secure additional revenue through contributing memberships but, however, this campaign has not been sufficiently aggressive to secure any marked response and the situation became such that this year the Board determined to enter on an aggressive campaign to raise the necessary revenue from this source. Unfortunately the condition in the cement industry during the early spring and summer months was such as to make it inexpedient to attempt to solicit contributing memberships, and this attempt was deferred until this fall when the business conditions became more favorable.

The Board has given careful consideration to the expenses of maintaining headquarters and performing the necessary work of the Association. The minimum annual expenditures, based upon past experience, including the cost of publishing one volume of the Proceedings has been about $9,500. To meet this expenditure we have been receiving from $5 dues about $3,500, from contributing memberships about $500, from the sale of Proceedings about $1,000, from advertising about $2,000 and from the Cement Show about $1,000, making the total receipts about $8,000. This

makes an estimated deficit of $1,500, which, together with the deficit existing on July 1, 1912, of $2,500, will make an estimated deficit on June 30, 1913, of $4,000, which does not include the cost of publishing the Proceedings of the present Convention, estimated to cost about $3,000, making the total deficit of $7,000.

About $2,000 has been raised in previous years through advertising. The Board feels, however, that in this campaign for contributing memberships, it is desirable to abandon the effort to secure revenue from advertisements. The Board has classified the various interests in the Associations, proportioning a definite amount to each, to be raised through contributing memberships, and has begun its campaign with the view of raising at least $6,000 per annum; these contributing memberships to continue until the revenue from dues shall be sufficient to meet the needs of the Association. In this work your President has already secured approximately one-half of this amount and your Board feels reasonably confident that he will be able to secure the remainder during the coming year.

The Board proposed at the last Convention to increase the annual dues from $5 to $10 but after an analysis of the letter ballot canvass of the opinion of the members, it recommended that in its judgment it was inexpedient to raise the dues at the present time. The Board is decidedly of the opinion that unless the various branches of the cement and concrete industry adequately support the Association and provide sufficient funds to meet its needs, the Association must discontinue. It is unreasonable to expect the officers, most of whom are engaged in private work, to give not only their time in the administration of its affairs but also to solicit funds for the support of the Association. The Association exists entirely for advancing the cement and concrete industry through the dissemination of information as to the proper use of cement. It is educational in its character and the Board feels that they can state without challenge that it has done more than any other single factor to steady and promote the best interests of the industry. Its recommendations for standard specifications and practice in the use of cement are being widely distributed and adopted by municipalities, engineers and others, and are very helpful in increasing the technical knowledge.

The cost of maintaining headquarters at the present time is

much lower than any similar organization in this country and is regarded by those qualified to speak as the lowest of any. Your Board feels reasonably confident that the members of the Association will use their best endeavors to increase the membership and in raising sufficient revenue through this and other contributing memberships to meet its obligations.

The Board believes that its plans for raising revenue will provide sufficient in excess of expenses so that the deficit may be made up within a reasonable time. The principal outstanding obligation of the Association is the bill of The John C. Winston Company, for printing the Proceedings, which amounts to $3,069.35. This bill has been carried from year to year and the present bill should be paid before any further obligation is incurred for the Proceedings of the Kansas City Convention. The Board wishes to convey to the Association the fact that the $5 membership, with the number of members now enrolled, does not supply sufficient revenue to meet the expenses and that the members can help to carry this burden through activity in building up the membership and through aiding in securing contributing memberships, so that the needed total revenue may be obtained.

Research Fund.—Many committees of the Association have found in the course of their work the necessity for carrying on special investigations and the research fund which was created a year ago to care for expenditures by the Committee on Reinforced Concrete has been maintained through voluntary contributions which have been raised for specific purposes.

The present program contemplates the making of tests of concrete columns 20 inches in diameter by 12 feet long and fire tests of concrete mullions. This work is invaluable to the industry and your Board feels that there should be at least $1,000 a year available for research work. Under the plan of the Board of Direction a definite allotment will be made yearly for this fund of which only a certain percentage will be available for investigative work during the year, the difference being added to the fund.

Had the Association been compelled to meet all the expenses of the tests made last year, the outlay would have approximated $8,000, but through the co-operation of Professor Arthur N. Talbot, of the Engineering Experiment Station of the University of Illinois, and of the owners and contractors of the buildings tested, the

Association was actually called upon to the extent of only $1,492.33. This work has given data on floor construction which is the most complete to be found anywhere. The Board wishes to express its appreciation of the co-operation of the University of Illinois and the private parties through whom these tests were possible.

Conclusion.—The Board feels that the influence of the Association is one of which every one should be proud and that its present position is one that should be maintained by the concrete industry. The importance of the work of the Association and the necessity for it is so evident that your Board feels confident that the financial needs will be met so that its development may be unhampered, thus making possible even greater accomplishments in the future.

APPENDIX.—MINUTES OF MEETINGS OF THE EXECUTIVE BOARD.

Meeting of the Elective Members of the Executive Board, Held March 16, 1912, at Hotel Baltimore, Kansas City, Mo.

Present: Richard L. Humphrey, President; Edward D. Boyer, First Vice-President; and Arthur N. Talbot, Second Vice-President.

It was decided to make the following appointments:

Treasurer, Henry C. Turner.

Secretary, Edward E. Krauss.

Sectional Vice-Presidents:

Building Blocks and Cement Products.—P. S. Hudson.

Education.—Logan Waller Page.

Exhibition.—H. S. Doyle.

Fireproofing.—Rudolph P. Miller.

Insurance.—W. H. Ham.

Measuring Concrete.—Robert A. Cummings.

Nomenclature.—Peter Gillespie.

Reinforced Concrete and Building Laws.—A. E. Lindau.

Roadways, Sidewalks and Floors.—C. W. Boynton.

Specifications and Methods of Tests for Concrete Materials.—Sanford E. Thompson.

Treatment of Concrete Surfaces.—L. C. Wason.

Adjournment.

Meeting of the Executive Board, Held April 25, 1912, at 11 Broadway, New York, N. Y.

Present: Richard L. Humphrey, President; Edward D. Boyer, First Vice-President; E. S. Larned, Third Vice-President; H. C. Turner, Treasurer; and Edward E. Krauss, Secretary.

On motion, the President, Mr. Richard L. Humphrey, was appointed official delegate to the Eighth International Congress of Applied Chemistry.

On motion, the President, Mr. Richard L. Humphrey, was appointed official delegate to the Sixth Congress of the International Association for Testing Materials.

On motion, the President, Mr. Richard L. Humphrey, the Second Vice-President, Professor Arthur N. Talbot, and Sectional Vice-President, Mr. A. E. Lindau, were appointed official representatives at the Sixteenth Annual Meeting of the National Fire Protection Association.

In view of the condition of the finances of the Association, it was decided not to accept at present the invitation to become a member of the British Fire Prevention Committee; the matter was held for future attention.

The request of the National Builders' Supply Association for copies of the standards for free distribution was considered and the President instructed to advise that the Association could not supply free pamphlets at this time.

The request of the New York State Library for copies of the Proceedings either by gift or at reduced prices, to replace those lost by fire was considered and it was decided, in view of these circumstances, to furnish the missing volumes at half price.

It was decided to grant the request of the Library of the University of Illinois, for an exchange of the Proceedings for the Bulletins of its Engineering Experiment Station, with the understanding that it furnish the Association a complete set of its Bulletins pertaining to concrete in exchange for future editions of the Proceedings.

It was decided not to authorize an exchange of the Proceedings for copies of the journal *The Cornell Civil Engineer*.

The storage of publications was referred to the President and Secretary with power to arrange at a monthly rental not to exceed $4.

The Secretary reported that 1,000 copies of Vol. VII were as yet unbound; it was decided to have 500 bound and to hold the remaining 500 copies until needed.

The request of the Editor of Engineering-Contracting for an additional copy of Vol. VII was granted.

An immediate campaign for the sale of publications to libraries, at an expense limited to $50, was authorized.

The President submitted a request for an investigation by the Association of Tufa concrete, and after a full consideration it was decided to be inexpedient to undertake the same at this time.

The following actions were taken on resolutions referred by the Eighth Annual Convention to the Executive Board.

It was decided to appoint a Committee on the Form of Standard Specifications and Recommended Practice and the President was requested to submit names for the chairman and members of this committee.

The appointment of a Committee on Standard Specifications for Highway Bridges and Culverts was referred to the President with power.

The Committee on Nomenclature was instructed to extend its work to cover the standardization of the size of drawings, symbols and the graphical representation of details.

The President was requested to appoint a Committee on Standard Specifications for Fence Posts.

The time and place for the Ninth Annual Convention was discussed and the President was authorized to arrange for this Convention to be held at the time of the New York Cement Show, provided the Cement Products Exhibition Company would donate to the Association the sum of $1,000.

The Treasurer presented a statement for the current fiscal year and the general question of finances was discussed.

A Committee on Finance was appointed consisting of Edward D. Boyer, Edward S. Larned and Henry C. Turner.

The Secretary presented the following tabulation of the postal card replies of the members of the Association in the matter of increase of dues:

	For.	Against.
Manufacturers, etc	38	21
Small users	43	51
Large contractors	30	30
Engineers	70	55
Architects	5	13
Press	5	1
Undetermined	4	4
	195	175

Adjournment.

MINUTES OF THE MEETING OF THE EXECUTIVE BOARD, HELD AT THE FORT PITT HOTEL, PITTSBURGH, PA., TUESDAY, DECEMBER 10, 1912.

Present: Richard L. Humphrey, President; Edward D. Boyer, First Vice-President; Edward S. Larned, Third Vice-President; William M. Kinney, L. C. Wason, Willis Whited, Sectional Vice-Presidents; and Edward E. Krauss, Secretary.

It was decided to accept the invitation from the H. J. Heinz Company to visit its plant Thursday, December 12, at 11 A. M.

In accordance with the provisions of the By-Laws the Executive Board divided the membership into the following districts for the ensuing year:

DISTRICT NO. 1.

Maine	Massachusetts
Vermont	Rhode Island
New Hampshire	Connecticut

DISTRICT NO. 2.

New York

DISTRICT NO. 3.

Pennsylvania	Delaware
New Jersey	Maryland

DISTRICT NO. 4.

Michigan	South Carolina
Indiana	Tennessee
Ohio	Georgia
Virginia	Florida
District of Columbia	Alabama
West Virginia	Mississippi
North Carolina	Kentucky

DISTRICT NO. 5.

Minnesota	Louisiana
Wisconsin	Oklahoma
Iowa	Indian Territory
Illinois	Texas
Missouri	Alaska
Arkansas	Foreign Members

DISTRICT NO. 6.

North Dakota	Kansas
South Dakota	Colorado
Montana	Utah
Washington	Nevada
Oregon	California
Idaho	Arizona
Wyoming	New Mexico
Nebraska	

In accordance with the requirements of the By-Laws the following appointments were made to be announced by the President at the first regular session of the Convention:

Committee on Nomination of Officers:

James O. Handy, Pittsburgh, Pa., Chairman.
John E. Conzelman, St. Louis, Mo.
William N. Denman, Springfield, Mass.
E. S. Hanson, Chicago, Ill.
W. W. Schouler, Newark, N. J.

Committee on Resolutions:

Fred K. Irvine, Chicago, Ill., Chairman.
Cloyd M. Chapman, New York, N. Y.
A. J. Maynard, State Farm, Mass.
Henry H. Quimby, Philadelphia, Pa.
R. J. Wig, Washington, D. C.

A committee consisting of Rudolph P. Miller and L. C. Wason was appointed to audit the accounts of the Secretary and Treasurer.

The Secretary reported the decease, on November 26, 1912, of Percy S. Hudson, Sectional Vice-President and Chairman of the Committee on Building Blocks and Cement Products. The Board adopted a minute on the death of Mr. Hudson.

The President presented a draft of the annual report of the Executive Board, which was fully discussed and revisions suggested to be presented at the next meeting of the Board.

The meeting then adjourned to re-convene on Wednesday, December 11, 1912.

MINUTES OF THE MEETING OF THE EXECUTIVE BOARD, HELD AT THE FORT PITT HOTEL, PITTSBURGH, PA., WEDNESDAY, DECEMBER, 11, 1912.

Present: Richard L. Humphrey, President; Edward D. Boyer, First Vice-President; Arthur N. Talbot, Second Vice-President; E. S. Larned, Third Vice-President; Henry C. Turner, Treasurer; Alfred E. Lindau, Rudolph P. Miller, L. C. Wason, Willis Whited, Sectional Vice-Presidents; Cloyd M. Chapman and Frank C. Wight, by invitation; Edward E. Krauss, Secretary.

The report of the Treasurer was approved.

The revised annual report of the Executive Board was presented and was adopted.

Adjournment.

REGISTER OF ATTENDANCE, NINTH CONVENTION.

Adams, J. C., Pittsburgh, Pa.
Affleck, B. F., Chicago, Ill.
Aldridge, E. V., Chicago, Ill.
Allen, Orrin T., Chicago, Ill.
Appleton Sewer Pipe Works, Appleton, Wis.
Armstrong, Joseph D., Pittsburgh, Pa.
Arp, C. K., Chicago, Ill.
Associated Metal Lath Manufacturers, Youngstown, Ohio.
Auryansen, F., Jamaica, N. Y.
Babcock, E. V., Pittsburgh, Pa.
Bates, P. H., Pittsburgh, Pa.
Beck, J. P., Chicago, Ill.
Boyer, Edward D., Catasauqua, Pa.
Boynton, C. W., Chicago, Ill.
Brown, Burtis S., Boston, Mass.
Brown, W. S., Pittsburgh, Pa.
Cabot, Samuel, Incorporated, Boston, Mass.
Caldwell, W. L., Canton, Ohio.
Carstanjen, Julius, Jr., Duisberg, Germany.
Cement Era, Chicago, Ill.
Chalfant, J. O.,
Chapman, Cloyd M., New York, N. Y.
Christopher, J. H., Ridgewood, N. J.
Chubb, J. H., Chicago, Ill.
Cohen, A. B., East Orange, N. J.
Concrete: Walter C. Boynton, Editor, Detroit, Mich.
Concrete Steel Company, Chicago, Ill.
Condron, T. L., Chicago, Ill.
Covelle, V. R.
Crawford Construction Company, Chicago, Ill.
Crescent Portland Cement Company, Wampum, Pa.
Cromar, A. J., Brantford, Ontario, Can.
Cummings, Robert A., Pittsburgh, Pa.
Danforth, G. H.,
Davis, Arthur P., Washington, D. C.
Davis, B. H., New York, N. Y.
Denman, W. N., Springfield, Mass.
Doney, F. H.,
Doyle, H. S., Chicago, Ill.
Engineering News, New York, N. Y.
Engineering Record, New York, N. Y.
Ferguson, John A., Pittsburgh, Pa.
Ferguson, L. R., Philadelphia, Pa.
Fowler, E. S., Chicago, Ill.
French & Company, Samuel H., Philadelphia, Pa.
Garland, Robert, Pittsburgh, Pa.
General Fireproofing Company, Youngstown, Ohio.
Gillespie, Peter, Toronto, Ontario, Canada.
Goehring, J. M.
Hall, R. F., Chicago, Ill.
Hall Concrete Products Company.
Ham, W. H., Boston, Mass.
Handy, J. O., Pittsburgh, Pa.
Hawley, W. C., Pittsburgh, Pa.
Healy, Clarence, Hoboken, N. J.
Higgs, Jr., J. A.,
Hiles, Elmer K., Pittsburgh, Pa.
Hines, Edward N., Detroit, Mich.
Hoeveler, W. A., Pittsburgh, Pa.
Hopper, James G., Ridgewood, N. J.
Humphrey, Richard L., Philadelphia, Pa.
Irvine, Fred K., Chicago, Ill.
Isenberg, G., New York, N. Y.
Johnson, T. H., Pittsburgh, Pa.
Kerr, J. P., Pittsburgh, Pa.
Kinney, William M., Pittsburgh, Pa.
Knickerbocker Portland Cement Company, New York, N. Y.
Kranz, H. J., Harbor Beach, Mich.
Krauss, Edward E., Philadelphia, Pa.

Landor, Edward J., Canton, Ohio.
Larned, E. S., Boston, Mass.
Lehigh Portland Cement Company, Allentown, Pa.
Lindau, Alfred E., St. Louis, Mo.
Luten, D. B., Indianapolis, Ind.
Macafee, S. B., Athens, Pa.
Magee, W. A., Pittsburgh, Pa.
Maynard, Arthur J., State Farm, Mass.
Mensch, L. J., Chicago, Ill.
Mesnard, H. W.
McArdle, P. J., Chicago, Ill.
McCall, W. A., Pittsburgh, Pa.
McCollum, Burton, Washington, D.C.
McDaniel, J. U. C., Chicago, Ill.
McKee, Logan, Pittsburgh, Pa.
McKee, W. E.,
Michigan Portland Cement Company, Jackson, Mich.
Miller, Rudolph P., New York, N. Y.
Moore, E. J., New York, N. Y.
Mott, W. E.,
Nordman, E. E.,
Norton, Henry A., Boston, Mass.
Page, L. W., Washington, D. C.
Parry, O. R., Philadelphia, Pa.
Peerless Brick Machine Company, Minneapolis, Minn.
Peters, O. S., Washington, D C.
Phillips Company, Akron, Ohio.
Phillips, J. W., Philadelphia, Pa.
Quick, Ray S., Pittsburgh, Pa.
Rackle & Sons Company, Cleveland, Ohio.
Rader, B. H., Pittsburgh, Pa.
Radford Publishing Company, Chicago, Ill.
Rauch, H., Philadelphia, Pa.
Raymond Brothers Company,
Reiland, A. F., Hoboken, N. J.
Roess, G. F., Oil City, Pa.
Rosa, E. B., Washington, D. C.
Schouler, W. W., Newark, N. J.
Schilling, Adolph, Philadelphia, Pa.
Smith, J. H., Columbus, Ohio.
Spencer, C. A., Glenshaw, Pa.
Sprague, N. S., Pittsburgh, Pa.
Stevenson, W. H., Pittsburgh, Pa.
Stocker Concrete Machine Works, Highland, Ill.
Stotz, Edward, Pittsburgh, Pa.
Stuart, J. L., Pittsburgh, Pa.
Swift, Willis G., Coshocton, Ohio.
Talbot, Arthur N., Urbana, Ill.
Talbot, K. H., Pittsburgh, Pa.
Thompson, Sanford E., Newton Highlands, Mass.
Tomkins, Ambrose, New York, N. Y.
Trimble, R. A., Pittsburgh, Pa.
Turner, Henry C., New York, N. Y.
Turner Concrete Steel Company, Philadelphia, Pa.
Voorhees, S. S., Washington, D. C.
Walters, J. D., Atlanta, Ga.
Wason, Leonard C., Boston, Mass.
Watson, C. D., Syracuse, N. Y.
Westinghouse, Church, Kerr & Company, New York, N. Y.
Whitted, Willis, Harrisburg, Pa.
Wig, Rudolph J., Washington, D. C
Wight, Frank C., New York, N. Y.
Wilkerson, T. J., Pittsburgh, Pa.
Wilkins, W. G., Pittsburgh, Pa.
Wilson, H. M., Pittsburgh, Pa.
Witherons, Irvin.,
Woodbury, S. S.,
Woōlson, Ira H., New York, N. Y.

SUBJECT INDEX.

Address.

Annual — by the President. The Use of Concrete in Mines. Richard L. Humphrey, 27.

Beams.

Tests to Determine Lateral Distribution of Stresses in Wide Reinforced Concrete —. W. A. Slater, 182.

Bins.

Reinforced Concrete Silos and Small Grain —. E. S. Fowler, 498.

Bridges (see Highway Bridges).

Building Laws.

Report of Committee on Reinforced Concrete and —. Alfred E. Lindau, Chairman, 112.

Cement. (see also Portland Cement).

The Constitution of Portland —. Some Results Obtained at the Experimental — Plant of the Bureau of Standards. P. H. Bates, 368.

The Progress of the Work of the Bureau of Standards in — and Concrete. Rudolph J. Wig, 380.

The Use of — on National Irrigation Works. Arthur P. Davis, 258.

Coloring.

The — and Texture of Concrete Products. Adolph Schilling, 491.

Committee.

Report of the — on Concrete Surfaces. L. C. Wason, Chairman, 233.

Report of the — on Measuring Concrete. Robert A. Cummings, Chairman, 348.

Report of the — on Reinforced Concrete and Building Laws. Alfred E. Lindau, Chairman, 112.

Report of the — on Reinforced Concrete Highway Bridges and Culverts. Willis Whited, Chairman, 412.

Report of the — on Roadways, Sidewalks and Floors. C. W. Boynton, Chairman, 427.

Report of the — on Specifications and Methods of Tests for Concrete Materials. Sanford E. Thompson, Chairman, 356.

Concrete. (See also Reinforced Concrete.)
Application of — in the Abolishment of Grade Crossings in the City of Philadelphia. James W. Phillips, 285.
— in Railroad Work. M. A. Long, 277.
— Elevators. Barney I. Weller, 338.
Effects of Electric Currents on —. E. B. Rosa, Burton McCollum, O. S. Peters, 45.
Recent Applications of — and Reinforced — in the City of Pittsburgh. N. S. Sprague, 307.
Report of the Committee on — Surfaces. L. C. Wason, Chairman, 233.
Report of the Committee on Specifications and Methods of Tests for — Materials. Sanford E. Thompson, Chairman, 356.
Some Recent Applications of — in Railroad Work. Frederick Auryansen, 293.
Standard Methods for the Measurement of — Work, 348.
Standard Specifications for — Curb and — Curb and Gutter, 442.
Standard Specifications for — Roads and Street Pavements, 455.
Standard Specifications for Plain — Floors, 431.
Tests of Waterproofing for —. Cloyd M. Chapman, 250.
The — Roads of Wayne County, Michigan. Edward N. Hines, 462.
The Coloring and Texture of — Products. Adolph Schilling, 491.
The Development of — Grain Elevator Construction. R. P. Durham, 326.
The Manufacture of — Products in Germany. Julius Carstanjen, Jr., 480.
The Progress of the Work of the Bureau of Standards in Cement and. — Rudolph J. Wig, 380.
The Use of — by the Bureau of Yards and Docks, U. S. Navy, H. R. Stanford, 266.
The Use of — in Mines. Richard L. Humphrey, 27.
Topical Discussion on — Floors, Sidewalks, Roads, Highway Bridges and Culverts, 404.
Vibrolithic — Pavements. R. C. Stubbs, 476.

Construction.
The Development of Concrete Grain Elevator —. R. P. Durham, 326.
The Present Status of Unit Methods of Reinforced Concrete —. John E. Conzelman, 218.
Principles of Design and Results of Tests and Girderless Floor — of Reinforced Concrete. T. L. Condron, 116.

Convention.
Register of Members in Attendance, Ninth —, 523.
Summary of the Proceedings of the Ninth Annual —, 15.

Culverts.
Topical Discussion on Concrete Floors, Sidewalks, Roads, Highway Bridges and —, 404.

Culverts (*Continued*)
Report of the Committee on Reinforced Concrete Highway Bridges and —. Willis Whited, Chairman, 412.

Curb.
Standard Specifications for Concrete — and Concrete — and Gutter, 442.

Design.
Principles of — and Results of Tests on Girderless Floor Construction of Reinforced Concrete. T. L. Condron, 116.
— of Rigid Frames in Steel and Reinforced Concrete. Sanford E. Thompson and Edward Smulski, 156.

Discussion.
— on Effects of Electric Currents on Concrete, 107.
— on Measurement of Actual Stresses in a Cantilever Flat Slab Reinforced Concrete Floor having Rectangular Panels, 151.
— on Recent Applications of Concrete and Reinforced Concrete in the City of Pittsburgh, 324.
— on Report of the Committee on Concrete Surfaces, 242.
— on Report of the Committee on Measuring Concrete, 355.
— on Report of the Committee on Reinforced Concrete Highway Bridges and Culverts, 417.
— on Report of the Committee on Specifications and Methods of Tests for Concrete Materials, 366.
— on Tests to Determine Lateral Distribution of Stresses in Wide Reinforced Concrete Beams, 215.
— on the Concrete Roads of Wayne County, Michigan, 470.
— on The Constitution of Portland Cement. Some Results Obtained at the Experimental Cement Plant of the Bureau of Standards, 378.
— on the Manufacture of Concrete Products in Germany, 488.
Topical — on Concrete Floors, Sidewalks, Roads, Highway Bridges and Culverts, 404.

Electric Currents. (See Concrete.)

Elevators.
Concrete —. Barney I. Weller, 338.
The Development of Concrete — Construction. R. P. Durham, 326.

Executive Board.
Annual Report of the —, 511.
Minutes of Meetings of the —, 518.

Flat Slab.
Measurement of Actual Stresses in a Cantilever — Reinforced Concrete Floor having Rectangular Panels. Arthur R. Lord, 127.

Floor.
Principles of Design and Results of Tests on Girderless — Construction of Reinforced Concrete. T. L. Condron, 116.
Measurement of Actual Stresses in a Cantilever Flat Slab Reinforced Concrete — having Rectangular Panels. Arthur R. Lord, 127.

Floors.
Topical Discussion on Concrete —, Sidewalks, Highway Bridges and Culverts, 404.
Report of the Committee on Roadways, Sidewalks and —. C. W. Boynton, Chairman, 427.
Standard Specifications for Plain Concrete —, 431.
Standard Specifications for Reinforced Concrete —, 437.

Grade Crossings.
Application of Concrete in the Abolishment of — in the City of Philadelphia. James W. Phillips, 285.

Grain Elevator. (See Elevators.)

Gravel. (See also Specifications.)
Sand and — Washing Plants. Raymond W. Dull, 393.

Gutter.
Standard Specifications for Concrete Curb and Concrete Curb and —, 442.

Highway Bridges.
Topical Discussion on Concrete Floors, Sidewalks, — and Culverts, 404.
Report of the Committee on Reinforced Concrete — and Culverts. Willis Whited, Chairman, 412.

Irrigation Works.
The Use of Cement on National —. Arthur P. Davis, 258.

Manufacture. (See Products.)

Materials. (See Specifications and Methods.)
Report of the Committee on Specifications and Methods of Tests for Concrete —. Sanford E. Thompson, Chairman, 356.

Measurement.
Standard Methods for the — of Concrete Work, 348.
— of Actual Stresses in a Cantilever Flat Slab Reinforced Concrete Floor having rectangular Panels. Arthur R. Lord, 127.

Measuring Concrete.
Report of the Committee on —. Robert A. Cummings, Chairman, 348.

Methods.
The Present Status of Unit — of Reinforced Concrete Construction. John E. Conzelman, 218.
Standard — for the Measurement of Concrete Work, 348.
Report of the Committee on Specifications and — of Tests for Concrete Materials. Sanford E. Thompson, Chairman, 356.

Mines.
The Use of Concrete in —. Richard L. Humphrey, 27.

National Association of Cement Users.
Charter, 9.
By-Laws, 10.
Officers, 5.
Sectional Committees, 6.
Past Officers, 8.
Summary of the Proceedings of the Ninth Annual Convention, 15.
Annual Report of the Executive Board, 511.
Minutes of Testings of the Executive Board, 518.
Register of Members in Attendance — Ninth Convention, 523.
List of Publications, 537.

Pavements. (See Roads.)
Standard Specifications for Concrete Roads and Street —, 455.
Vibrolithic Concrete —. R. C. Stubbs, 476.

Portland Cement. (See also Cement.)
Standard Specifications for — Sidewalks, 449.
Standard Specifications for — Stucco, 233.
The Constitution of —. Some Results Obtained at the Experimental Cement Plant of the Bureau of Standards. P. H. Bates, 368.

Presidential Address. (See Address.)

Proceedings. (See Convention.)

Products.
The Manufacture of Concrete — in Germany. Julius Carstanjen, Jr., 480.
The Coloring and Texture of Concrete —. Adolph Schilling, 491.

Publications.
List of —, 537.

Railroad Work. (See Concrete.)
Concrete in —. M. A. Long, 277.
Some Recent Applications of Concrete in —. Frederick Auryansen, 293.

Reinforced Concrete. (See also Concrete.)
Design of Rigid Frames in Steel and —. Sanford E. Thompson and Edward Smulski, 156.
Measurement of Actual Stresses in a Cantilever Flat Slab — Floor having Rectangular Panels. Arthur R. Lord, 127.
Principles of Design and Results of Tests on Girderless Floor Construction of —. T. L. Condron, 116.
Recent Applications of Concrete and — in the City of Pittsburgh. N. S. Sprague, 307.
Report of the Committee on — and Building Laws. Alfred E. Lindau, Chairman, 112.
Report of the Committee on — Highway Bridges and Culverts. Willis Whited, Chairman, 412.
— Silos and Small Grain Bins. E. S. Fowler, 498.
Standard Specifications for — Floors, 437.
Tests to Determine Lateral Distribution of Stresses in Wide — Beams. W. A. Slater, 182.
The Present Status of Unit Methods of — Construction. John E. Conzelman, 218.

Report. (See Committee.)
Annual — of the Executive Board, 511.
— of the Committee on Concrete Surfaces. L. C. Wason, Chairman, 233.
— of the Committee on Measuring Concrete. Robert A. Cummings, Chairman, 348.
— of the Committee on Reinforced Concrete and Building Laws. Alfred E. Lindau, Chairman, 112.
— of the Committee on Reinforced Concrete Highway Bridges and Culverts. Willis Whited, Chairman, 412.
— of the Committee on Roadways, Sidewalks and Floors. C. W. Boynton, Chairman, 427.
— of the Committee on Specifications and Methods of Tests for Concrete Materials. Sanford E. Thompson, Chairman, 356.

Results of Tests.
Principles of Design and — on Girderless Floor Construction of Reinforced Concrete. T. L. Condron, 116.
The Constitution of Portland Cement. Some — Obtained at the Experimental Cement Plant of the Bureau of Standards. P. H. Bates, 368.

Rigid Frames.
Design of — in Steel and Reinforced Concrete. Sanford E. Thompson and Edward Smulski, 156.

Roads. (See Highways and Pavements.)
The Concrete — of Wayne County, Michigan. Edward N. Hines, 462.

Roads (*Continued*).
Topical Discussion of Concrete Floors, Sidewalks, —, Highway Bridges and Culverts, 404.
Standard Specifications for Concrete — and Street Pavements, 455.

Roadways. (See Committee.)

Sand. (See also Specifications.)
— and Gravel Washing Plants. Raymond W. Dull, 393.

Sidewalks.
Report of the Committee on Roadways, — and Floors, C. W. Boynton, Chairman, 427.
Standard Specifications for Portland Cement —, 449.
Topical Discussion on Concrete Floors, —, Roads, Highway Bridges and Culverts, 404.

Silos.
Reinforced Concrete — and Small Grain Bins. E. S. Fowler, 498.

Specifications.
Report of the Committee on — and Methods of Tests for Concrete Materials. Sanford E. Thompson, Chairman, 356.

Standard Methods (See also Methods.)
— for the Measurement of Concrete Work, 348.

Standard Specifications.
— for Concrete Curb and Concrete Curb and Gutter, 442.
— for Concrete Roads and Street Pavements, 455.
— for Plain Concrete Floors, 431.
— for Portland Cement Sidewalks, 449.
— for Portland Cement Stucco, 233.
— for Reinforced Concrete Floors, 437.

Steel. (See Rigid Frames.)

Street Pavements. (See Standard Specifications.)
Standard Specifications for Concrete Roads and —, 455.

Stresses. (See Reinforced Concrete.)
Measurement of Actual — in a Cantilever Flat Slab Reinforced Concrete Floor having Rectangular Panels, Arthur R. Lord, 127.
Tests to Determine Lateral Distribution of — in Wide Reinforced Concrete Beams. W. A. Slater, 182.

Stucco.
Standard Specifications for Portland Cement —, 233.

Surfaces.
Report of the Committee on Concrete —. L. C. Wason, Chairman, 233.

Tests.
Report of the Committee on Specifications and Methods of — for Concrete Materials. Sanford E. Thompson, Chairman, 356.
The Principles of Design and Results of — on Girderless Floor Construction of Reinforced Concrete. T. L. Condron, 116.
— to Determine Lateral Distribution of Stresses in Wide Reinforced Concrete Beams. W. A. Slater, 182.
— of Waterproofing for Concrete. Cloyd M. Chapman, 250.

Texture.
The Coloring and — of Concrete Products. Adolph Schilling, 491.

Unit Methods.
The Present Status of — of Reinforced Concrete Construction. John E. Conzelman, 218.

Washing Plants.
Sand and Gravel —. Raymond W. Dull, 393.

Waterproofing.
Tests of — for Concrete. Cloyd M. Chapman, 250.

Vibrolithic Concrete.
— Pavements. R. C. Stubbs, 476.

AUTHOR INDEX.

Arp, C. K.
Discussion, 246.

Auryansen, Frederick.
Some Recent Applications of Concrete in Railroad Work, 293.
Discussion, 108, 109.

Bates, P. H.
The Constitution of Portland Cement. Some Results Obtained at the Experimental Cement Plant of the Bureau of Standards, 368.
Discussion, 378, 379.

Boyer, Edward D.
Discussion, 109, 242, 246.

Boynton, C. W.
Report of Committee on Roadways, Sidewalks and Floors, 427.
Discussion, 248, 378, 379, 489.

Brett, Allen.
Discussion, 405, 409.

Bullard, George P.
Discussion, 245.

Carstanjen, Julius, Jr.
The Manufacture of Concrete Products in Germany, 480.
Discussion, 488, 489, 490.

Chapman, Cloyd M.
Tests of Waterproofing for Concrete, 250.
Discussion, 107, 108, 109, 110, 243, 246, 247, 248.

Christopher, Joseph H.
Discussion, 243.

Cohen, A. B.
Discussion, 324.

Condron, T. L.
Principles of Design and Results of Tests on Girderless Floor Construction of Reinforced Concrete, 116.
Discussion, 151, 152, 153, 154, 155, 216.

Conzelman, John E.
The Present Status of Unit Methods of Reinforced Concrete Construction, 218.

Cummings, Robert A.
Report of Committee on Measuring Concrete, 348.
Discussion, 151, 152, 355.

Davis, Arthur P.
The Use of Cement on National Irrigation Works, 258.

Dull, Raymond W.
Sand and Gravel Washing Plants, 393.

Durham, R. P.
The Development of Concrete Grain Elevator Construction, 326.

Ferguson, Lewis, R.
Discussion, 242, 246, 473, 474.

Fowler, E. S.
Reinforced Concrete Silos and Small Grain Bins, 498.

Godfrey, Edward.
Discussion, 489.

Hines, Edward N.
The Concrete Roads of Wayne County, Michigan, 462.
Discussion, 470, 473, 474.

Humphrey, Richard L.
The Use of Concrete in Mines, 27.
Discussion, 109, 243, 244, 404, 407, 408, 409, 410, 421, 488, 489.

Larned, Edward S.
Discussion, 425, 474, 490.

Lindau, Albert E.
Report of Committee on Reinforced Concrete and Building Laws, 112.
Discussion, 151, 154, 216.

Long, M. A.
Concrete in Railraod Work, 277.

Lord, Arthur R.
Measurement of Actual Stresses in a Cantilever Flat Slab Reinforced Concrete Floor having Rectangular Panels, 127.
Discussion, 152, 153, 154, 155, 215, 216.

Lovis, A. M.
Discussion, 418.

Luten, Daniel B.
Discussion, 419, 423.

Maynard, Arthur J.
Discussion, 409, 470.

McCollum.
Effects of Electric Currents on Concrete, 45.
Discussion, 107, 110, 111.

McMaster, H. B.
Discussion, 108, 242, 244.

Mensch, L. J.
Discussion, 421, 474.

Miller, Rudolph P.
Discussion, 355.

Peters, O. S.
Effects of Electric Currents on Concrete, 45.

Phillips, James W.
Applications of Concrete in the Abolishment of Grade Crossings in the City of Philadelphia, 285.

Quimby, Henry H.
Discussion, 417.

Rosa, E. B.
Effects of Electric Currents on Concrete, 45.

Schilling, Adolph.
The Coloring and Texture of Concrete Products, 491.

Schouler, W. W.
Discussion, 244, 247, 248, 406, 407, 408, 409, 473.

Shattuck, Harold B.
Discussion, 422.

Slater, W. A.
Tests to Determine Lateral Distribution of Stresses in Wide Reinforced Concrete Beams, 182.
Discussion, 215.

Smulski, Edward.
Design of Rigid Frames in Steel and Reinforced Concrete, 156.

Sprague, N. S.
Recent Applications of Concrete and Reinforced Concrete in the City of Pittsburgh, 307.
Discussion, 324.

Stanford, H. R.
The Use of Concrete by the Bureau of Yards and Docks, United States Navy, 266.

Stubbs, R. C.
Vibrolithic Concrete Pavements, 476.

Talbot, Arthur N.
Discussion, 423.

Talbot, Kenneth H.
Discussion, 404,406,407,409.

Thompson, George V.
Discussion, 247.

Thompson, Sanford E.
Design of Rigid Frames in Steel and Reinforced Concrete, 156.
Report of Committee on Specifications and Methods of Tests for Concrete Materials, 356.
Discussion, 152, 215, 216, 249, 366.

Wason, Leonard C.
Report of Committee on Concrete Surfaces, 233.

Watson, Charles D.
Discussion, 488, 489, 490.

Weller, Barney I.
Concrete Elevators, 338.

Whited, Willis
Report of Committee on Reinforced Concrete Highway Bridges and Culverts, 412.
Discussion, 107, 324, 406, 408, 417, 421, 425, 426.

Wig, Rudolph J.
The Progress of the Work of the Bureau of Standards in Cement and Concrete, 380.
Discussion, 108, 110, 244.

LIST OF PUBLICATIONS.

A. PROCEEDINGS.

VOLUME I.

Coloring of Concrete—J. P. Sherer.
Testing of Cement Blocks—John R. Allen.
The Dry Mixture of Concrete—A. L. Goetzmann.
Practical Work of Constructing Sidewalks—A. T. Gridley.
The Waterproofing of Concrete Blocks—G. B. Kirwan.
Waterproofing Concrete—W. H. Finley.
Cement Posts—J. A. Mitchell.
Mortar Sand—J. C. Hain.
Proceedings of the First Convention.
Discussion:
Coloring of Concrete.
Testing of Cement Blocks.
The Dry Mixture of Concrete.
The Practical Work of Constructing Sidewalks.
Register of Members in Attendance—First Convention.
List of Exhibitors First Cement Show.

VOLUME II.

Annual Presidential Address—Richard L. Humphrey.
Concrete Aggregates—Sanford E. Thompson.
Discussion.
Cement and Building Construction—C. A. P. Turner.
Discussion.
Concrete Building Blocks—S. B. Newberry.
Discussion.
Some Notes on Reinforced Concrete—A. L. Johnson.
The Use of Cement and Concrete for Farm Purposes—S. M. Woodward.
What the Cement Users Owe to the Public—R. W. Lesley.
Cement Block Architecture—Louis H. Gibson.
Waterproofing—J. L. Mothershead, Jr.
The Causes of Failure in the Concrete Block Business—O. U. Miracle.
The Selection of Portland Cement to be Used in the Manufacture of Concrete Blocks—Richard K. Meade.
Discussion.
Legislation Concerning the Use of Cement in New York City—R. P. Miller.
Discussion.
Legislation on Concrete Building Blocks—Will J. Scoutt.

Hair Cracks, Crazing or Map Cracks on Concrete Surfaces—Albert Moyer.
Report of Committee on Fireproofing and Insurance—E. T. Cairns.
Discussion.
Report of Committee on Art and Architecture—Charles D. Watson.
Discussion.
Machinery for Cement Users—W. W. Benson.
Topical Discussion on Cement Products.
Observations on the Testing and Use of Portland and Natural Cements—E. S. Larned.
Manufactured Stone—J. C. McClenahan.
Discussion.
The Manufacture and Use of Concrete Piles—Henry Longcope.
The Use of Salt in Concrete Sidewalk Construction—George L. Stanley.
Discussion.
Topical Discussion on Sidewalk Laying.
General Discussion on Cement and Concrete.
Summary of Proceedings of the Second Convention.
Register of Members in Attendance—Second Convention.
List of Exhibitors Second Cement Show.
Minutes of the Meetings of the Executive Board.
List of Members.

VOLUME III.

Summary of Proceedings of the Third Convention.
Annual Address by the President—The Successes and Failures of Cement Construction—Richard L. Humphrey.
Report of Committee on Sidewalks, Streets and Floors—George L. Stanley, Chairman.
Discussion.
Cement Sidewalk Paving—Albert Moyer.
Discussion.
Reinforced Concrete—W. K. Hatt.
Discussion.
Forms for Concrete Construction—Sanford E. Thompson.
Discussion.
Topical Discussion on the Manufacture of Concrete Blocks.
Selecting the Proportions for Concrete—William B. Fuller.
Discussion.
Report of Committee on Testing of Cement and Cement Products—E. S. Larned, Chairman.
Discussion.
The Artistic Treatment of Concrete—A. O. Elzner.
Concrete Surfaces—H. H. Quimby.
Discussion.
Treatment of Concrete Surfaces—Linn White.
Discussion.

Report of Committee on Art and Architecture—Charles D. Watson, Chairman.
Discussion.
Discussion on Streets, Sidewalks and Floors.
Concrete Blocks—Harmon H. Rice.
Discussion.
Tests of Building Blocks—R. D. Kneale.
Report of Committee on Machinery for Cement Users—J. F. Angell, Chairman.
Topical Discussion on the Manufacture of Cement Blocks.
Joint Meeting Sections on Testing Cement and Cement Products and on Concrete Blocks and Cement Products.
Waterproofing Cement Mortars and Concretes—The Asphalt Mastic Method—H. Wiederhold.
Discussion.
Waterproofing Cement Mortars and Concretes—The Elastic *vs.* the Rigid Method—Edward W. DeKnight.
Discussion.
Waterproofing Cement Mortars and Concretes—The Dry Compound Method—R. R. Fish.
Discussion.
Waterproofing Cement Mortars and Concretes—The Liquid Method—G. G. Fry.
Waterproofing Cement Mortars and Concretes—The Hydrocarbon Paint Method—S. J. Binswanger.
Discussion.
Report of Committee on Fireproofing and Insurance—E. T. Cairns, Chairman.
Report of Committee on Laws and Ordinances—H. C. Henley, Chairman.
Notes on the Investigation of Cement Mortars and Concretes in the United States Geological Survey Laboratories, at St. Louis, Mo.—Richard L. Humphrey.
Register of Members in Attendance—Third Convention.
List of Exhibitors Third Cement Show.
Report of the Executive Board.
Minutes of the Meetings of the Executive Board.
List of Members.

VOLUME IV.

Summary of Proceedings of the Fourth Convention.
Annual Address by the President—The Year's Progress in the Cement Industry and the Work of the Association—Richard L. Humphrey.
Tests of Reinforced Concrete Hollow Tile Floor Spans—W. K. Hatt.
Discussion.
Methods and Costs of Reinforced Concrete Construction with Separately Molded Members—William H. Mason.
Discussion.

The Necessity of Continuity in the Steel Reinforcement of Concrete Structures—E. P. Goodrich.
Discussion.
The Unit *vs.* The Loose Bar System of Reinforced Concrete Construction—Emile G. Perrot.
Discussion.
Factory-Made Concrete—Charles D. Watson.
Co-operation—What it is and What it can Accomplish—Robert W. Lesley.
Proportioning and Mixing Cement Mortars and Concrete—Leonard C. Wason.
Discussion.
Specifications for Cement Sidewalks—C. W. Boynton.
Report of the Committee on Streets, Sidewalks and Floors—George. L. Stanley, Chairman.
Discussion.
Standard Specifications for Portland Cement Sidewalks.
Report of Committee on Testing Cement and Cement Products—E. S. Larned, Chairman.
Standard Specifications for Cement Hollow Building Blocks.
Progress in Manufacture and Use of Cement Building Blocks—J. W. Pierson.
Topical Discussion on the Manufacture of Hollow Cement Blocks.
The Value of Sand in Concrete Construction—E. S. Larned.
General Discussion on the Use of Cement.
Report of the Committee on Machinery for Cement Users—Mentor Wetzstein, Chairman.
Report of the Committee on Fireproofing and Insurance—William M. Bailey, Chairman.
Report of the Committee on Laws and Ordinances—H. C. Henley, Chairman.
Report of the Committee on Art and Architecture—Charles D. Watson, Chairman.
Concrete from the Architect's Point of View—E. B. Green.
Exposed Selected Aggregates in Monolithic Concrete Construction—Albert Moyer.
Reinforced Concrete from the Contractor's Point of View—H. H. Fox.
Discussion.
Metal Forms in Reinforced Concrete Construction—W. L. Caldwell.
Discussion.
Manipulation of Forms in Concrete Construction—J. F. Swinnerton.
Reclamation of Arid Land—E. T. Perkins.
Waterproofing Cement Structures—James L. Davis.
Progress in the Investigation of Cement Mortars and Concretes in the Structural Materials Testing Laboratories of the United States Geological Survey, St. Louis, Mo.—Richard L. Humphrey.
Register of Members in Attendance—Fourth Convention.
List of Exhibitors Fourth Cement Show.
Report of the Executive Board.
Minutes of Meetings of the Executive Board.
List of Members.

VOLUME V.

Summary of Proceedings of the Fifth Convention.

Annual Address by the President. The Progress of the Association and the Necessity for Fireproof Construction—Richard L. Humphrey.

Cost of Concrete Construction as applied to Buildings—Leonard C. Wason. Discussion.

Comparative Cost of Reinforced Concrete Buildings—Emile G. Perrot.

Monolithic Concrete Wall Buildings—Methods, Construction and Cost—Robert Aiken.

Cold Storage Warehouses of Reinforced Concrete Construction—J. P. H. Perry. Discussion.

Evolution of Reinforcement for Concrete—Harry F. Porter.

Concrete Applied to Dwelling House Construction—Ross F. Tucker.

Waterproofing—Various Applications and Comparative Costs—T. Hugh Boorman.

Methods of Attaching Shafting and Machinery in Reinforced Concrete Buildings—William M. Bailey.

Report of Committee on Art and Architecture—Albert Moyer, Secretary.

Decorative Concrete Stone—Frederick A. Norris.

Unburnable Homes—Their Artistic and Architectural Possibilities—Benjamin A. Howes.

The Construction and Cost of Small Concrete Houses—C. R. Knapp.

The Importance and Cost of Cement Testing—W. Purves Taylor.

The Availability of Concrete for Bridges—Its Cost and Durability—H. H. Quimby.

Cost of Reinforced Concrete Bridges, Especially with Regard to Maintenance—E. P. Goodrich.

Reinforced Concrete Retaining Walls—A. E. Lindau. Discussion.

The Applicability and Comparative Cost of Concrete and Reinforced Concrete for Subway Construction—Charles M. Mills. Discussion.

The Advantages of Reinforced Concrete for Railway Construction—B. H. Davis.

Concrete Piles—Forms, Advantages and Cost as Compared with Wooden Piles—C. W. Gaylord.

Value and Cost of Steel Centers for the Construction of Culverts and Bridges—L. H. Scott.

The Present and Future of the Cement Block—Its Manufacture, Its Availability, Its Cost—J. Augustine Smith.

Value and Cost of Steam Curing of Cement Hollow Blocks—F. S. Phipps.

Progress in the Use of Metal Forms with Comparative Costs—W. L. Caldwell. Discussion.

Cost and Value of Concrete Pavements—J. H. Chubb.

Report of Committee on Streets, Sidewalks and Floors—W. W. Schouler, Chairman.

Standard Specifications for Portland Cement Sidewalks.

Proposed Specifications for Concrete Roads.
Discussion.
Report of Committee on Insurance, Laws and Ordinances—W. H. Ham, Chairman.
Part I, on Insurance.
Part II, on Building Regulations.
Proposed Standard Building Regulations for Reinforced Concrete.
Discussion.
Report of Committee on Reinforced Concrete—Sanford E. Thompson, Chairman.
Discussion.
Report of Committee on Testing Cement and Cement Products—E. S. Larned, Chairman.
Standard Specifications for Cement Hollow Building Blocks.
Discussion.
Topical Discussion on Reinforced Concrete.
Report of the Executive Board.
Minutes of Meetings of the Executive Board.
Register of Members in Attendance—Fifth Convention.
List of Exhibitors Fifth Cement Show.
List of Members.
Geographical Distribution of Members.

VOLUME VI.

Summary of Proceedings of the Sixth Convention.
Annual Address by the President. The Use of Concrete in Europe—Richard L. Humphrey.
Reinforced Concrete Columns—Peter Gillespie.
Discussion.
Proposed Methods for the Reinforcement of Concrete Compression Members—Robert A. Cummings.
Discussion.
Longitudinal Reinforcement in Concrete Columns—Sanford E. Thompson.
Concrete for Maritime Structures—Chandler Davis.
The Essential Qualities and the Application of Concrete to Timber Structures in Sea Water for the Purpose of Increasing their Permanency—Ralph Barker.
Discussion.
The Efficiency and Cost of Concrete for the Preservation of Piles Exposed to Sea Water—C. C. Horton.
Results of Experiments upon Effect of Sea Water on the Tensile Strength of Various Mixtures of Cement and Sand—Cloyd M. Chapman.
Discussion.
Laying Concrete under Water—Detroit River Tunnel—Olaf Hoff.
Discussion.
Application of Concrete in Barge Canal Work—R. S. Greenman.
Discussion.

The Concrete Groined Arch in Filter and Covered Reservoir Construction—T. H. Wiggin.

A Simple Method of Computing the Strength of Flat Reinforced Concrete Plates—Angus B. MacMillan.

Discussion.

A Method for Long Span, Light Floor, Reinforced Concrete Construction with Comparative Cost—Emile G. Perrot.

The Preparation of Concrete—From Selection of Materials to Final Deposition—Harry Franklin Porter.

Report of Committee on Concrete and Reinforced Concrete—Alfred E. Lindau, Chairman.

Discussion.

Topical Discussion on Concrete and Reinforced Concrete.

Report of Committee on Building Laws and Insurance—William H. Ham, Chairman.

Part I.—Building Laws.

Standard Building Regulations for the Use of Reinforced Concrete.

Part II.—Insurance.

Topical Discussion on Insurance.

Report of Committee on Specifications for Fireproofing—Rudolph P. Miller, Chairman.

Concrete Construction with Separately Molded Members and Costs—Charles D. Watson.

Discussion.

Report of Committee on Exterior Treatment of Concrete Surfaces—L. C. Wason, Chairman.

Topical Discussion on the Exterior Treatment of Concrete Surfaces.

Reinforced Concrete for the Small House—C. R. Knapp.

Inexpensive Homes of Reinforced Concrete—Milton D. Morrill.

Discussion.

The Use of Concrete in Farm Buildings from a Sanitary Point of View—S. Cunningham, Jr.

Development of Concrete Road Construction—Fred R. Charles.

Discussion.

Notes on the Use and Cost of Concrete Blocks in Roadway Construction—George C. Wright.

Report of Committee on Roadways, Sidewalks and Floors—C. W. Boynton, Chairman.

Standard Specifications for Portland Cement Sidewalks.

Standard Specifications for Concrete Road and Street Pavements.

Standard Specifications for Portland Cement Curb and Gutter.

Topical Discussion on Roadways, Sidewalks and Floors.

Report of Committee on Specifications for Cement Products—W. P. Anderson, Chairman.

Proposed Standard Specifications for Architectural Concrete Blocks.

Proposed Standard Specifications for Plain Concrete Drain Tile.

Topical Discussion on Cement Products.

Waterproofing Concrete Without Altering Its Appearance—Cloyd M. Chapman.
Discussion.
Cost and Advantages of Concrete Drain Tile—J. H. Libberton.
Discussion.
Essentials in Cement Hollow Block Construction—Ernest B. McCready.
Installation and Operation of a Steam Curing Plant—F. S. Phipps.
Discussion.
Report of the Executive Board.
Minutes of Meetings of the Executive Board.
Register of Members in Attendance—Sixth Convention.

VOLUME VII.

Summary of Proceedings of the Seventh Convention.
Official Delegates, Seventh Annual Convention.
Annual Address by the President. Some Fallacies in Methods of Fireproofing—Richard L. Humphrey.
The National Fire Protection Association and Its Work—W. H. Merrill.
Some Thermal Properties of Concrete—Charles L. Norton.
Discussion.
Report of the Committee on Fireproofing—Rudolph P. Miller, Chairman.
Report of the Committee on Insurance—William H. Ham, Chairman.
An Incident of the Value of Concrete in Reducing the Cost of Insurance—Emile G. Perrot.
Discussion.
The Henry Hudson Memorial Bridge—William H. Burr.
Concrete Filled Steel Arches—H. H. Quimby.
Construction Problems in Concrete Bridges—Walter M. Denman.
A Test of a Flat Slab Floor in a Reinforced Concrete Building—Arthur R. Lord.
A Discussion of the Basis of Design for Reinforced Concrete Floor Slabs—Arthur R. Lord.
Discussion.
The Calculation of Reinforced Concrete Flat Plates Supported by Four Columns—L. J. Mensch.
Analysis of Results of Load Tests on Panels of Reinforced Concrete Buildings—Emile G. Perrot.
The Web Reinforcement of Concrete Beams—John Stephen Sewell.
Discussion.
Notes on Web Reinforcement—Peter Gillespie.
Problems Encountered in the Construction of the Aziscohos Storage Dam—Seth A. Moulton.
Concrete Construction. Testimony of the Roman Forum and Some Modern Examples—Alfred Hopkins.
The Use of Reinforced Concrete for Hospitals and Similar Structures—R. A. McColloch.
Reinforced Concrete School Buildings—John T. Simpson.

Use of Reinforced Concrete in San Francisco and Vicinity—John B. Leonard.
The Taylor Concrete Coal Breaker—R. D. Richardson.
Moving Forms for Reinforced Concrete Storage Bins—James MacDonald.
The Use of Reinforced Concrete in Sea Water—Raymond Baffrey.
Discussion.
Some Special Uses of Concrete in Mining—George S. Rice.
Report of the Committee on Reinforced Concrete and Building Laws—Alfred E. Lindau, Chairman.
Topical Discussion on Concrete and Reinforced Concrete.
Comparative Cost and Maintenance of Various Types of Building Construction—J. P. H. Perry.
Discussion.
Regulation Relative to the Design and Construction in Reinforced Concrete. Hungarian Society of Engineers and Architects.
The Measurement of Concrete Construction—Robert A. Cummings.
The Use of Compressed Air in the Handling of Mortar and Concrete—G. L. Prentiss.
The Spouting of Concrete—Fred E. Engstrum.
The Preparation and Handling of Concrete—H. M. Cryder.
Report of the Committee on Treatment of Concrete Surfaces—L. C. Wason, Chairman.
Proposed Standard Specifications for Portland Cement Stucco.
Standard Specifications for Scrubbed Concrete Surface.
Discussion.
Topical Discussion on the Treatment of Concrete Surfaces.
Dustless Concrete Floors—L. C. Wason.
Discussion.
Tufa Concrete—Joseph B. Lippincott.
Discussion.
Regulations Governing the Uniform Transportation and Examination of Cements and Trass—Hungarian Society of Engineers and Architects.
The Relation of the Lime Content of Cement to the Durability of Concrete—Henry S. Spackman.
Discussion.
The Insulation of Concrete, Especially below Grade—Edward W. DeKnight.
The Effect of Electrolysis on Metal Imbedded in Concrete—Cloyd M. Chapman.
Discussion.
Waterproofing with Water—Cloyd M. Chapman.
Discussion.
The Waterproofing of Tunnels—A. H. Harrison.
Topical Discussion on Waterproofing of Sewers.
Report of the Committee on Roadways, Sidewalks and Floors—C. W. Boynton, Chairman.
Some New Methods in Sidewalk and Curb and Gutter Construction—Jerome B. Landfield.
Topical Discussion on Roadways, Sidewalks and Floors.

Report of the Committee on Building Blocks and Cement Products—P. S. Hudson, Chairman.
Proposed Standard Specifications for Concrete Drain Pipe and Tile.
Proposed Standard Specifications for Architectural Concrete Stone.
Cement Drain Tile Plant—Layout and Operation—C. M. Powell.
Discussion.
Additional Notes on Steam Curing Plants—F. S. Phipps.
General Discussion on Cement Products.
Report of the Executive Board.
Minutes of Meetings of the Executive Board.
Register of Members in Attendance—Seventh Convention.

VOLUME VIII.

Summary of Proceedings of the Eighth Annual Convention.
Annual Address by the President. European Practice in Concrete Construction—Richard L. Humphrey.
Report of the Committee on Reinforced Concrete and Building Laws—A. E. Lindau, Chairman.
Discussion.
The Testing of Reinforced Concrete Buildings Under Load—W. A. Slater.
The Design of Concrete Flat Slabs—F. J. Trelease.
Discussion.
The Practical Design of Reinforced Concrete Flat Slabs—Sanford E. Thompson.
Discussion.
The Design of Concrete Grain Elevators—E. Lee Heidenreich.
Discussion.
Report of the Committee on Measuring Concrete—Robert A. Cummings, Chairman.
Proposed Standard Methods for the Measurement of Concrete Work.
Concrete Retaining Walls—John M. Meade.
Discussion.
Reinforced Concrete Piles—Robert A. Cummings.
The Handling of Concrete in the Construction of the Panama Canal—S. B. Williamson.
Use of Concrete in the Fourth Avenue Subway, Brooklyn, N. Y.—Frederick C. Noble.
The Use of Reinforced Concrete in Hypochlorite Water Purification Works—Walter M. Cross.
Design and Construction of the Estacada Dam—H. V. Schreiber.
Unit Cost of Reinforced Concrete for Industrial Buildings—C. S. Allen.
Reinforced Concrete Convention Hall at Breslau, Germany—S. J. Trauer.
The Suitability of Concrete for Gas Holder Tanks—Herbert W. Alrich.
Protection of Steel in Catskill Aqueduct Pipe Siphons—Alfred D. Flinn.
A Fireproof School of Concrete—Theodore H. Skinner.

The Present Status of Unit Concrete Construction—James L. Darnell.
Discussion.
Report of Committee on Specifications and Methods of Tests for Concrete Materials—Sanford E. Thompson, Chairman.
Aggregates for Concrete—William M. Kinney.
Discussion.
Field Inspection and Testing of Concrete—G. H. Bayles.
Comparative Tests of the Strength of Concrete in the Laboratory and in the Field—Rudolph J. Wig.
Discussion.
The Necessity for Field Tests of Concrete—Fritz Von Emperger.
Discussion.
Report of the Committee on Treatment of Concrete Surfaces—L. C. Wason, Chairman.
Discussion.
Cement Coatings—F. J. Morse.
Discussion.
Review of the Present Status of Iron Portland Cement—P. H. Bates.
Marine or Iron Ore Cements—Herman E. Brown.
Iron Ore Cement—Arthur E. Williams.
Discussion on Iron Ore Cement.
Flat Slab Concrete Bridges—William H. Finley.
Concrete Highway Bridges—Walter Scott Gearhart.
Concrete Bridges—Daniel B. Luten.
Discussion.
Report of Committee on Roadways, Sidewalks and Floors—C. W. Boynton, Chairman.
Standard Specifications for Portland Cement Sidewalks.
Standard Specifications for Concrete Roads and Street Pavements.
Standard Specifications for Concrete Curb and Concrete Curb and Gutter.
Standard Specifications for Plain Concrete Floors.
Standard Specifications for Reinforced Concrete Floors.
Discussion.
An Improved Concrete Pavement—E. W. Groves.
Cement Paving as Constructed at Mason City, Iowa—F. P. Wilson.
Discussion.
Report of the Committee on Building Blocks and Cement Products—P. S. Hudson, Chairman.
Recommended Practice for Plain Concrete Drain Tile.
Recommended Practice for Concrete Architectural Stone, Building Block and Brick.
Standard Specifications for Concrete Architectural Stone, Building Block and Brick.
Standard Building Regulations for the Use of Concrete Architectural Stone, Building Block and Brick.
Method of Testing Cement Pipe—Arthur N. Talbot and Duff A. Abrams.
Advantages and Durability of Cement Sewer Pipe—Gustave Kaufman.

The Manufacture and Use of Cement Drain Tile—Charles E. Sims.
Discussion.
Modern Methods of Manufacturing Concrete Products—Robert F. Havlik.
Notes on Reinforced Concrete Telegraph Poles—George Gibbs.
Concrete Fence Posts—W. J. Towne.
Concrete Fence Posts—L. J. Hotchkiss.
Discussion.
Report of the Executive Board.
Minutes of Meetings of the Executive Board.
Register of Members in Attendance—Eighth Convention.

B. STANDARDS.

1. Standard Specifications for Cement. With Appendix on Standard Methods of Testing and Chemical Analysis.
2. Standard Specifications for Portland Cement Sidewalks.
3. Standard Specifications for Cement Hollow Building Blocks.
4. Standard Building Regulations for the Use of Reinforced Concrete.
5. Standard Specifications for Concrete Road and Street Pavements.
6. Standard Specifications for Portland Cement Curb and Gutter.
7. Recommended Practice for Reinforced Concrete.
8. Standard Specifications for Scrubbed Concrete Surface.
9. Standard Recommended Practice for Concrete Drain Tile.
10. Standard Recommended Practice for Concrete Architectural Stone, Building Block and Brick.
11. Standard Specifications for Concrete Architectural Stone, Building Block and Brick.
12. Standard Building Regulations for the Use of Concrete Architectural Stone, Building Block and Brick.
13. Standard Specifications for Plain Concrete Floors.
14. Standard Specifications for Reinforced Concrete Floors.
15. Standard Specifications for Portland Cement Stucco.
16. Standard Methods for the Measurement of Concrete.

C. REPORTS.

1. On Insurance.
2. On Treatment of Concrete Surfaces.
3. On Tests of Reinforced Concrete Buildings.

NORTHEASTERN UNIVERSITY LIBRARIES
3 9358 00859883 8

National Association of 9 1913
AUTHOR
Cement Users. Proceedings.
TITLE

DISCARDED BY 9
GORDON McKAY LIBRARY 1913

GORDON MCKAY LIBRARY
PIERCE HALL
HARVARD UNIVERSITY

Please sign your name and address on this card, and deposit in box provided.
This book may be kept

OVER NIGHT

NORTHEASTERN UNIVERSITY LIBRARIES
3 9358 00859883 8